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FOUNDATIONS

FOR

BRIDGES AND BUILDINGS

BY

ROLAND P. DAVIS, S.B., M.C.E.

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From

FOUNDATIONS OF BRIDGES AND BUILDINGS

By

HENRY S. JACOBY and ROLAND P. DAVIS

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PREFACE

In writing the chapter on cofferdams an attempt has been made to classify, give examples of, and to point out the advantages of, the different types of cofferdams placed under various conditions. In an art which is based almost entirely upon precedent there is often too marked a tendency to copy methods used by others without carefully analyzing the conditions to see what type is best adapted in any given instance. It is true that in many instances any particular type has but few advantages over some other type, but, nevertheless, given certain physical conditions obtaining at the site, there is always some one type of cofferdam which will prove most satisfactory and economical.

The pneumatic caisson process of placing foundations affords an example of the value of applying scientific principles in the development of an art. The wooden type of caisson is the most widely used in this country and for this reason more space has been devoted to it than to the other types.

One of the problems largely unsolved as yet is the question of caisson disease. The two articles on this subject give a careful résumé of our present knowledge of this subject.

Because of the relatively low cost of shallow footings the subject of spread foundations is an important one. In the chapter on this subject all types which have proven successful and economical are described. Although any rational analysis of the stresses in a spread footing must be more or less approximate, yet only through such a design, coupled with good judgment, can an economical footing be developed.

TABLE OF CONTENTS

CHAPTER VI.—Cofferdams

Art. 64. The Cofferdam Process
65. Earth Cofferdams
66. Wooden Sheet-Pile Cofferdams
67. Single Wall with Guide Piles
68. Sheet Piling Supported by Frames
69. Sheet Piling Supported by Cribs
70. Steel Sheet-Pile Cofferdams
71. Self-Supporting Steel Sheet-Pile Cofferdams
72. Crib Cofferdams
73. Movable Cofferdams
74. Miscellaneous Types
75. Puddle and Leakage
76. Design of Cofferdams
77. Choice of Type

CHAPTER VIII.—Pneumatic Caissons for Bridges

Art. 92. The Pneumatic Process
93. Caisson Roof Construction
94. Sides of Working Chamber
95. Details of Cutting Edge
96. Bracing of Caisson
97. Crib Construction
98. Cofferdam Construction
99. Pneumatic Caissons of Concrete
100. Pneumatic Caissons of Metal
101. Cylinder Pier Caissons
102. Combination Cylinder Caissons

CHAPTER IX.—Pneumatic Caissons for Bridges (Con.).

103. Shafts and Air-Locks
104. Design of Caissons
105. Building and Placing the Caisson
106. Sinking the Caisson
107. Removing Spoil from Working Chamber
108. Concreting the Air Chamber
109. Rate of Sinking
110. Frictional Resistance
111. Physiological Effects of Compressed Air
112. Prevention of Caisson Disease

CHAPTER XV.—Spread Foundations

Art. 149. General Considerations
150. Early Types of Footings
151. Modern Types of Spread Foundations
152. Construction of I-Beam Grillage
153. Design of I-Beam Grillage
154. Design of Double-Column Footings
155. Distribution of Pressure on Base
156. Steel Grillage Foundations
157. Design of Reinforced-Concrete Spread Foundations
158. Design of Reinforced-Concrete Column Footings
159. Concrete Spread Foundations

the section. The values of the section modulus for the commercial sizes of steel sheet piles may be obtained from the manufacturers. The corresponding width to be used in computing the bending moment per pile is the distance center to center of interlock when assembled.

The design of sheet-piling to resist earth pressure in which the material has more or less cohesion is not on a basis that is entirely satisfactory. The conditions vary so widely and often the material penetrated in any locality occurs in layers of different density or character that it is well to make the design so as to be on the safe side. Some engineers design all sheet-piling for hydrostatic pressure, increased by 50 percent or more for wet slippery material.

CHAPTER VI

COFFERDAMS

Art. 64. The Cofferdam Process

When, for some purpose, it is desired to exclude the water and expose a portion of the bottom of a river, lake, or other body of water, a structure called a cofferdam is employed. This cofferdam is a temporary structure, practically water-tight and large enough to provide adequate room for working.

Defined, a cofferdam is a temporary structure used for the purpose of excluding the water from a given site, or area, either wholly or to such a degree that with a reasonable amount of pumping the permanent substructure may be built within it in the open air, or that other work may be accomplished.

The building of the permanent substructure may include pile driving, placing grillages, building piers and abutments, etc., while other work may include the construction of dams, removal of sunken vessels, etc. Where the ground is saturated with water, cofferdams are sometimes used in placing foundations for buildings.

Cofferdams are usually built in place. They may be self-contained or may depend for strength on the natural bottom, as is the case where guide piles are used. Bracing may be used to resist the lateral pressure against the walls.

To obtain water-tightness the sides of the cofferdam must be tight and the soil on which the cofferdam rests must be impervious. If the latter condition does not exist, either the sides of the cofferdam must extend through the pervious material to an impervious stratum or else a layer of concrete must be spread over the bottom inside the cofferdam and allowed to harden before pumping is begun. Absolute water-tightness is seldom sought, it being cheaper to pump a moderate

amount of leakage than to go to the heavy expense of building a structure that will not leak. The cofferdam should be so designed that the combined cost of construction, maintenance and pumping shall be a minimum.

To depths of from 20 to 30 feet the cofferdam process will prove the best and cheapest method of founding bridge piers and abutments, but for depths greater than 30 feet, owing to the difficulty of properly bracing the cofferdam against the pressure of the water, as well as preventing heavy leakage, some other method is usually preferable. Cofferdams over 50 feet deep have been used in a few instances.

Cofferdams may be constructed of earth, timber, steel or concrete. They may be divided into five general classes: earth, sheet pile, crib, movable and miscellaneous cofferdams. These classes will be described separately in the following articles.

ART. 65. EARTH COFFERDAMS

Of the five classes the earth cofferdam is the oldest in origin and simplest in construction. Its use is usually limited to shallow water with low velocities of current. It is made of a bank of earth placed around the site to be enclosed, and of a thickness sufficient to furnish the required stability and to keep the leakage down to a small amount. The earth bank should be carried up 2 or 3 feet above the water-level with a width of at least 3 feet at the top, and with side slopes corresponding to the natural slope of the material. The embankment should preferably be composed of a mixture of clay and sand or gravel, but if clay is scarce the bank may be composed of sand with a clay wall in the center.

The amount of embankment may be somewhat reduced by using one or two rows of sheet-piling, in which case the cofferdam may resemble more or less closely the sheet-pile cofferdam described in later articles. As to whether in any given case the cofferdam should be classed as an earth or sheet-pile cofferdam will depend upon whether or not stability and water-tightness depend primarily upon the earth filling.

Where the depth of water is not more than 4 or 5 feet and the velocity of the current would wash away loose material, cofferdams may be made of ordinary canvas bags about half filled with a mixture of clay and sand. It is important that the bags shall be but partially filled for otherwise they will not pack together closely.

A modern and up-to-date use of the earth cofferdam is found in the construction of the cofferdams of the West Neebish Channel of the St. Mary's River. In some places the depth of the water was far too great for the economical use of earth cofferdams and was justified here only by the extremely favorable conditions that obtained for placing the earth. Two subsidiary cofferdams were first constructed across the channel about midway between the main ones in order to stop the current and divert the flow to another course. [1] "These temporary dams were about 1000 feet apart at the site of the channel and extended across the river from the mainland to the island, varying in direction to suit the contours of the river bed. They were built in 2 to 7 feet of water flowing 3 to 6 miles an hour. The construction of these dams stopped the flow of water in the West Neebish Channel of the river, that the main cofferdams could be built in still water, and also laid bare a part of the site of the channel about 1000 feet long. In building these temporary dams, which varied from 4 to 10 feet in height, broken stone and rock were dumped from scows on the line of the dams until the force of the current was broken and the rock fill carried above the water. Sandy clay was then brought in and dumped on the upstream side of these rock embankments in order to silt up the openings and produce water-tight dams."

The main cofferdams which unwatered the 8600-foot section of the work were structures of unusual size. The upstream cofferdam was 1900 feet long and was built in water from 2 to 18 feet in depth. [1] "This cofferdam has a minimum width of 8 feet at the top, which is 7 feet above the water, and has side slopes on the water side of about 1 on $1\frac{1}{2}$, and of about 1 on

[1] Engineering Record, vol. 56, page 112, Aug. 3, 1907.

2 on the other side. The other main cofferdam is 8600 feet downstream from this one. It has a total length of 2600 feet, and in plan is arched slightly downstream against the water on that side of it. This cofferdam was built in water from nothing to 26 feet deep; it has a minimum width of 12 feet at the top, which is 6 feet above the water; its side toward the water is built on an average slope of 1 on 2, and the one on the other side of 1 on $2\frac{1}{2}$.

"The construction of the upstream main cofferdam was started soon after the current of the river had been broken by the temporary dams. Sandy clay and mud excavated by the

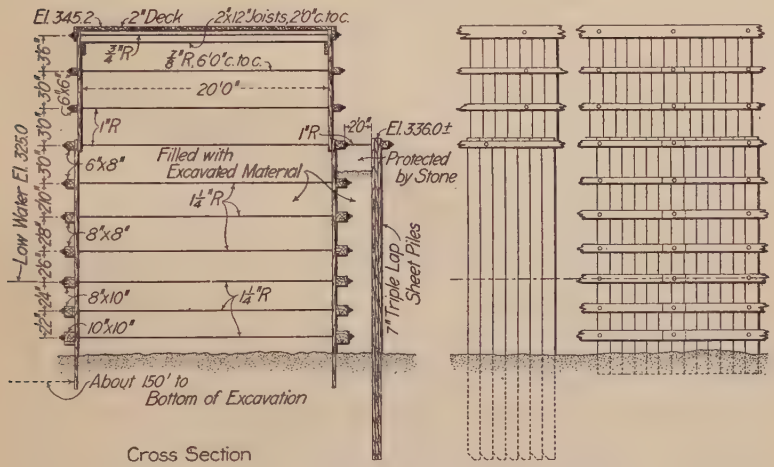


FIG. 65a.—Sheeting for Earth Cofferdam on the Ohio River.

dredges at work on the adjacent sections of the channel were brought to the site in bottom-dump scows and deposited in place. When the banks thus formed had been carried up until the bottom-Pump scows would operate no longer, the materials were loaded on flat-deck scows, and handled from these to place in the embankment by a clam-shell bucket on a derrick scow."

French engineers have made extensive use of the earth cofferdam for work on their various canals. In some work on the Meuse Canal, described in Annales des Ponts et Chaussees,

1896, page 539, gravel ranging in size from about 1 to 4 inches—being the residue, after the sand was used, of material dredged from the canal bottom—was employed. Water-tightness was obtained by placing a layer of tan-bark over the water face. In some cases the head of water on the cofferdam was as much as 9 feet.

In place of earth, cofferdams are sometimes made of fascines. The cofferdam for a concrete dam at Milford, Conn., was made by forming brush into mats, which were sunk by loading with rocks, the layers of brush and stone alternating. To give water-tightness a layer of earth was placed over the upstream side.

Fig. 65*a* illustrates the cross-section of the earth cofferdam with sheeting used in the construction of the Ohio River Lock

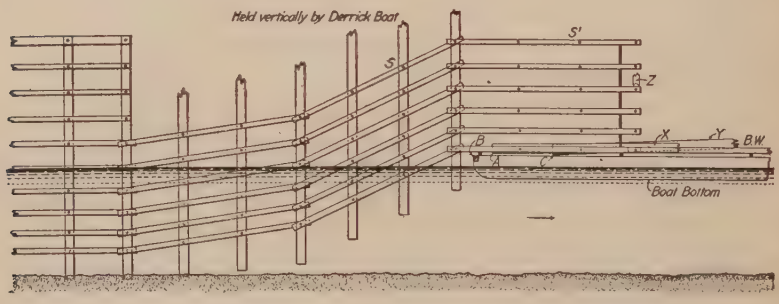


FIG. 65*b*.—Method of Constructing Ohio River Cofferdam.

and Dam 48, where the bottom was composed of sand. To break the current a line of sheet-piling was first driven. Frames were then placed by a boat as shown in Fig. 65*b* and connected to the sheet-piling. Vertical planking was placed against the frames and the interior then filled wirh dredged material. Gravel was placed along the outside of the sheet-piling up to its top and on a slope of about 45 degrees; the space between the sheet-piling and sheeting was also filled with earth, and finally sand was placed against the inside wall of sheeting up to the elevation of the sheet-piling tops. This sand had a very gentle slope, running approximately 100 feet before reaching the elevation of the bottom of the sheeting.

ART. 66. WOODEN SHEET-PILE COFFERDAMS

The sheet-pile type may be considered as the standard form of cofferdam. It consists of rows of sheet-piling, usually not more than two, extending around the site to be enclosed. The piling is held in place in various ways as described in the following articles. The sheet-piling serves the function of giving water-tightness to the structure, and to this end some form of intermeshing or interlocking piling is always employed. Strength to resist the pressure of the water outside is furnished by guide piles, frames, or cribs, in addition to a large amount of internal bracing. The sheet-piling may be of wood or steel; at the present time (1914) the use of various forms of steel piling is rapidly increasing.

Where it is possible to drive piles some distance into the soil the sheet-piling is best supported by vertical guide piles and horizontal wales. The latter will not only furnish a guide for the sheet-piling while being driven but will also add strength to the cofferdam, thus decreasing the amount of internal bracing necessary.

DOUBLE WALL WITH GUIDE PILES.—Fig. 66*a* shows the details of this type of cofferdam. It is composed of vertical guide piles, horizontal waling and cap timbers, vertical sheet piles and a puddle filling. Rods are usually put in near the top to connect each pair of guide piles in order to prevent the filling from spreading the walls apart. If the top of the cofferdam is but slightly above water-level, struts are often placed near and parallel to the tie rods, serving to hold the two walls apart.

The bearing piles are driven more deeply into the earth than the sheet piles, the aim being to drive them far enough to develop the full transverse strength of the pile when acting as a free cantilever above the earth. The sheet-piling should be driven to a fairly impervious stratum to prevent leakage under the cofferdam. The space between the walls should be filled with earth, preferably an intimate mixture of sand and clay or gravel and clay, to form a puddle (Art. 75), which will mate-

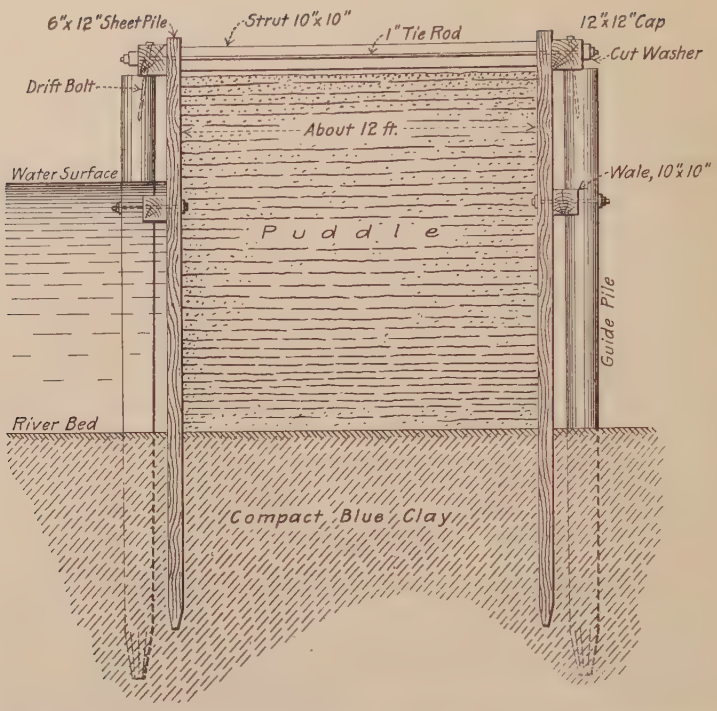


FIG. 66*a*.—Section of the Double Wall of a Cofferdam Showing Puddle Chamber.

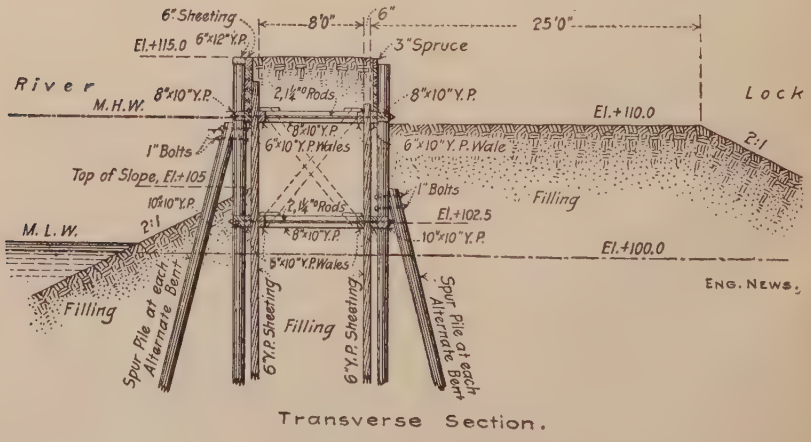


FIG. 66*b*.—Details of Double Wall of Sheet-pile Cofferdam, Charles River, Boston, Mass.

rially assist the sheet-piling in making the cofferdam water-tight. This puddle should be placed in thin layers and thoroughly tamped in a damp state. Before placing the same it will usually be advisable to dredge out the soft material on the bottom to an impermeable stratum. This puddle filling, in addition to promoting water-tightness, will materially strengthen the structure. Clay is often banked around the outside of the cofferdam to safeguard it further against leakage.

The cofferdam should have its puddle chamber wide enough to develop the required strength, furnish water-tightness, and afford sufficient space for placing machinery, gangways, etc. One rule for the width of unbraced cofferdams is to make it equal to the height above the ground up to 10 feet, and when the height is greater than this, make the width 10 feet plus one-third the height in excess of 10 feet. The design of sheet-piling is considered in Art. 63.

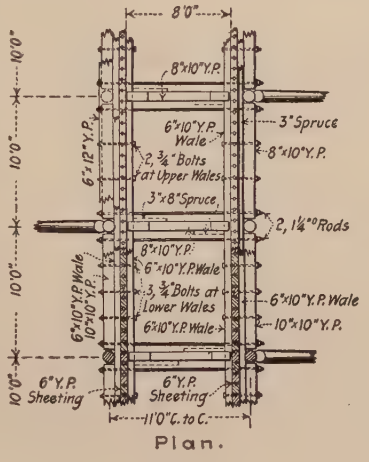


FIG. 66c.

A well-designed cofferdam of the double-wall type was used in the construction of the locks for the Charles River Dam, Boston, Mass., where the length was about 625 feet and the width about 250 feet, surrounding an area of approximately 4 acres. The maximum depth of water on the outside at low water was 20 feet. As shown in Fig. 66*b* and *c*, the cofferdam consisted of two rows of guide piles 11 feet apart, with piles spaced 10 feet on centers, which through wales supported 6-inch splined and grooved sheet-piling.

The guide piles were of spruce, 45 feet long, and each alternate pile was braced by a batter or spur pile. The sheet-piling was of yellow pine with spruce splines and was 38 feet in length. The remainder of the details are clearly shown in the diagrams. A filling of sand and clay was placed around both the inside and

outside of the cofferdam as well as in the puddle chamber. On the inside it had a width of 25 feet at the top and then sloped down on a 2 on 1 slope, thus making virtually a combination pile and earth cofferdam. Although probably not an economical form of cofferdam for ordinary use, yet in a case like this where the filling was permanent construction, it made an admirable structure to withstand the 37-foot head, which was approximately the maximum height of high water above the bottom of the lock masonry.

ART. 67. SINGLE WALL WITH GUIDE PILES

Where the space available for the cofferdam is restricted or where the area of the site to be enclosed is small and the head of water not great, a cofferdam having a single wall is preferable to the double-wall type. Other conditions being the same the former type will require more bracing than the latter, but in many cases this will prove cheaper than the extra wall.

Figs. 67*a* and *b* show the details of cofferdams used for the rectangular and pivot piers for the Illinois Central Railroad bridge across the Tennessee River at Gilberstville, Ky., both of which are standard types for single-wall cofferdams of moderate size. Before placing these cofferdams the bottom of the river was dredged down to about 17 feet below low water to hard gravel. Cofferdam guide piles were then driven and 10×12-inch wales bolted to the outside, after which 9×12-inch triple-lap sheet-piling was driven against the latter, penetrating the gravel from 4 to 6 feet. The piers were founded on bearing piles driven from 16 to 20 feet into the gravel and cut off 2 feet above the bottom before the cofferdam was placed.

Before pumping out the water a 3-foot layer of concrete was placed on the bottom, thus preventing leakage of water beneath the cofferdam; later it served as a cap for the bearing piles. The bracing, which is clearly shown in the illustrations, was placed as the water was pumped out. The octagonal cofferdam was braced by annular trusses which, by their arch-like action, proved to be a very rigid form of bracing, and yet offered no

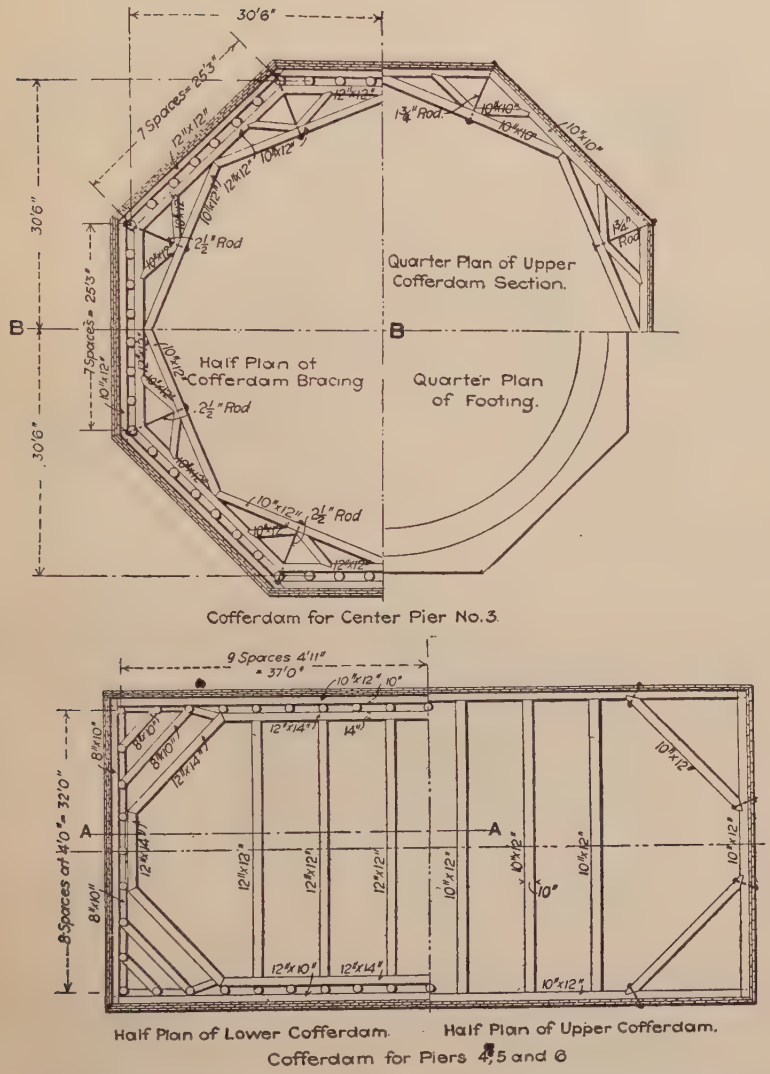


FIG. 67*a*.—Cofferdams with Single Walls of Timber Sheet Piling Supported by Wales and Guide Piles, for Piers of Illinois Central Railroad Bridge over Tennessee River, at Gilbertsville, Ky. See also Fig. 62*a*.

obstruction to the work of building the piers, which were of concrete. The forms for these piers were braced against the trusses.

A good example of a very large and high single-wall sheet-pile cofferdam, very strongly braced, is illustrated in Fig. 67*c*, this structure being used to found the pier of a lift bridge for

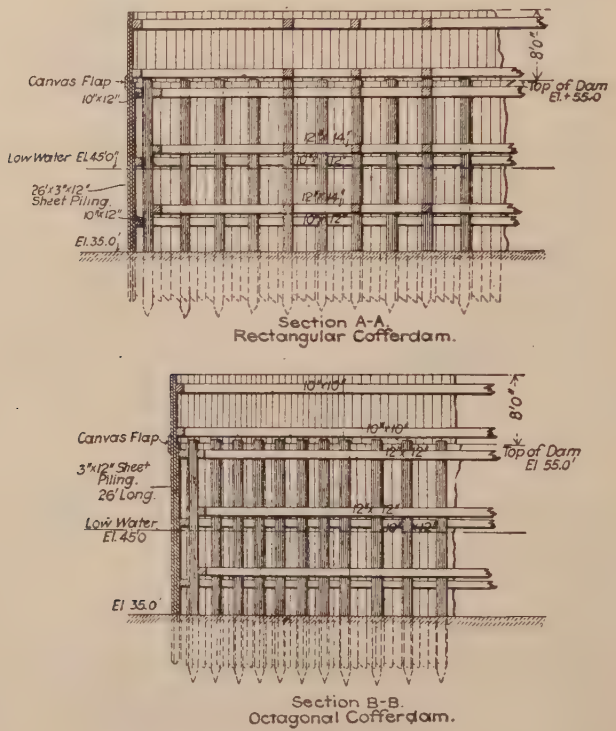


FIG. 67*b*.—Elevation of Cofferdam Walls.

the Chicago Terminal Transfer Railroad. Two sides of the cofferdam were on land, one in water, and the other two partly in water and partly on land.

A row of guide piles, from 6 to 8 feet apart and 40 feet long, were first driven. [1] "Six tiers of inside and outside waling pieces were bolted to these piles, and on the land side 370 6×12-inch

[1] Engineering Record, vol. 50, page 636, Nov. 26, 1904.

sheet piles 34 feet long were driven between the outer wales, and 6×12-inch horizontal guide pieces at the surface of the

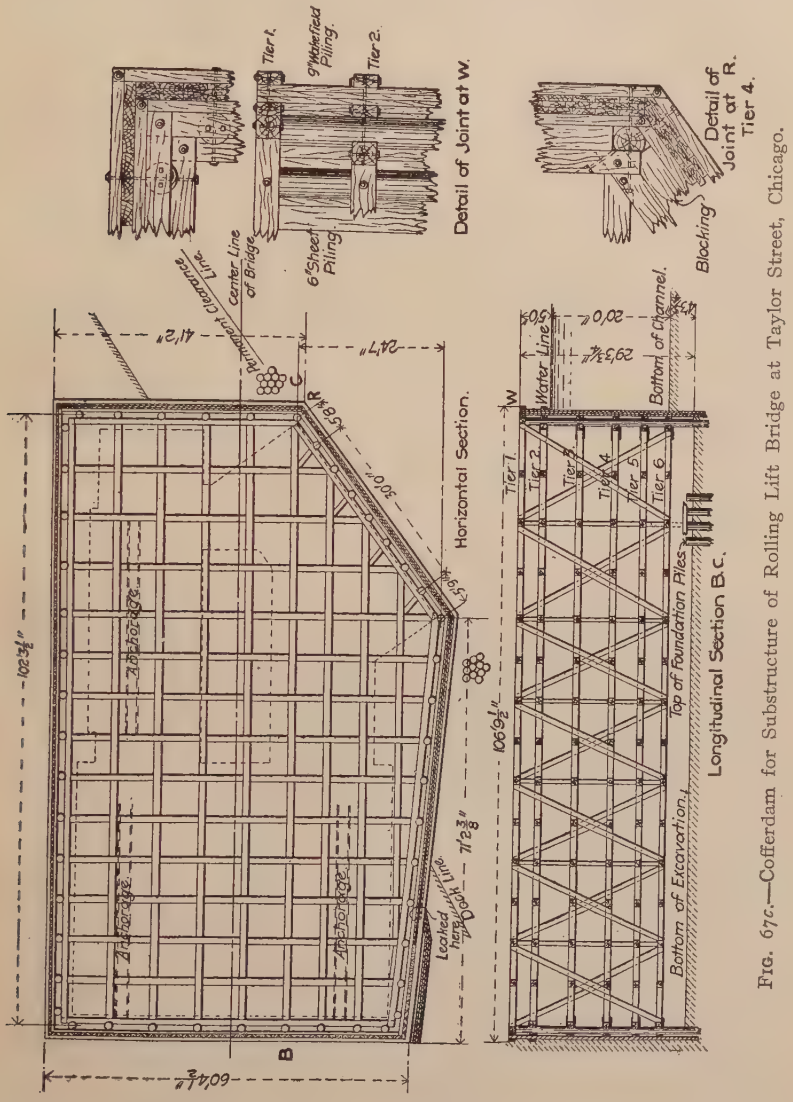


FIG. 67c.—Cofferdam for Substructure of Rolling Lift Bridge at Taylor Street, Chicago.

ground and 4 feet below it. On the water sides 274 34-foot Wakefield piles 9 inches thick, made of 3×12-inch planks, were driven in the same manner.

"The piles were driven as the excavation progressed inside of the cofferdam, and at the same time rows of transverse and longitudinal 12×12-inch horizontal braces, about 6 and 8 feet apart on centers and from 4 to 6 feet apart vertically, were set with their ends engaging the round piles on the center lines of the walls. At intersections these braces were supported on 8×8-inch vertical timbers; one of them was continuous and the other was cut to clear it, with the square ends abutting against the sides of the first piece and spliced across it with two side fish plates. . . . The inside wales were of 12×12-inch timber (except in the upper two tiers, where 8×16-inch timber was used because it was conveniently available from the contractor's stock), all of them being lapped and halved at intersections. The outside wales were uniformly 6×12 inches. The round pile caps and the two upper rows of wales on the water side were made of 9×13-inch timber. All wales were bolted through the round piles, and the oblique joint in the Wakefield piling was tied by bolts through both faces.

"In the longest dimension of the cofferdam, the six tiers of horizontal struts in each longitudinal line were divided into seven panels by the vertical posts supporting them at the intersections of alternate transverse braces. Each panel thus formed on three of the long lines and one short line was X-braced with 2×10-inch planks, spiked to the longitudinal struts at all intersections and overlapping in the centers of the panels, as shown in the longitudinal sectional elevation. Six lines of similar bracing were provided for the transverse struts, but varied from that in the longitudinal direction in that the upper and lower pieces of the bracing overlapped each other by the width of the space between two transverse struts, thus increasing the amount of bracing and the rigidity at a point half way between the top and bottom of the cofferdam."

ART. 68. SHEET-PILING SUPPORTED BY FRAMES

Where the nature of the bottom is such that piles cannot penetrate the same it is necessary to employ a frame to hold the

sheet-piling in place. These frames are usually built on shore, floated to the site, and sunk. Where piers are to be built under an existing bridge it is sometimes possible to suspend the frame from the bridge.

SINGLE-WALL TYPE.—At the site of the bridge piers of the Chicago, Milwaukee & St. Paul Ry. near Kilbourn, Wis.,

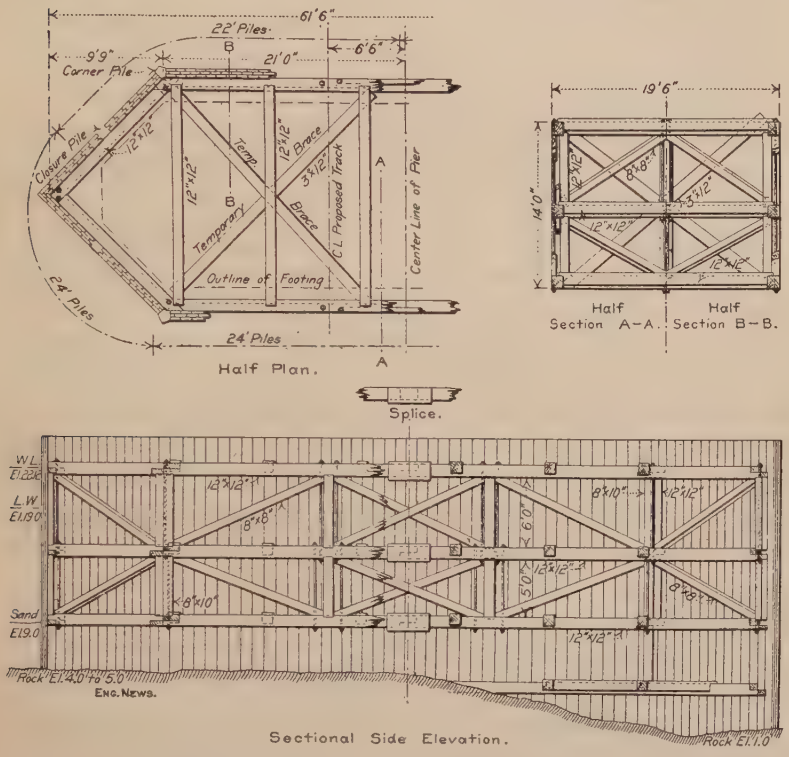


FIG. 68*a*.—Cofferdam for Pier of Chicago, Milwaukee, and St. Paul Railway, Kilbourn, Wis.

only a few feet of sand covered the rock bottom on which the piers were to rest. As the channel was narrow and the current swift it was essential that the current be obstructed as little as possible, and for this reason the single-wall type was chosen in preference to that having a double wall. On account of the slight depth of sand, guide piles could not be used and so

recourse was had to a frame. As shown in Fig. 68 *a*, the cofferdam had V-shaped ends to diminish the force of the current against the structure and was held in place by wire guys anchored to the rocks on the sides of the river. The frame, the details of which are shown in the illustration, was sunk by weighting with scrap rails. The covering consisted of 9×12-inch Wakefield sheet-piling; in driving this piling care was taken to broom the lower ends to give a close fit to the irregular rock surface.

To aid in giving water-tightness to the structure canvas was placed around the outside of the cofferdam, and was so arranged that the lower part rested flat on the river bed for a distance of 8 feet out from the dam, while the upper part extended above water-level. The lower part of the canvas was first weighted down with iron rails and sand bags to make it fit closely, after which about fifty car loads of gravel were placed upon it. As the water was pumped out the structure was thoroughly braced as shown, but on building the pier this bracing was removed and the cofferdam walls braced against the pier.

One of the largest and highest cofferdams ever built of wood was of the single-wall sheet-pile-on-frame type, and was used for the Mare Island Dry Dock No. 2. For a complete description of this structure see Engineering Record, vol. 57, page 428, April 4, 1908.

The cofferdam was approximately 150 by 800 feet in plan and the maximum head of water on it was 48 feet. The framework and bracing consisted of five horizontal courses of transverse and longitudinal timbers, the timbers of each course being connected to those of the adjacent courses by posts, the whole structure being built as one unit which rested on bearing piles previously driven and sawed off under water. These longitudinal and transverse rows were 12 feet apart on centers. In the bottom course all timbers were 16×16 inches in section, while those in the next two courses were 14×14 inches, with 12×12-inch timbers for the two upper courses. The rangers, *i.e.*, the horizontal pieces forming the frame proper which holds the sheet-piling in position, were 20×24 inches in section for the

bottom course and 12×12 inches for the top course, the other courses having intermediate sizes between these limits. The distance between courses was approximately 10 feet. In addition to the members mentioned, a large amount of bracing in both horizontal and vertical planes was used.

The sheet-piling units were formed of two 12×12-inch timbers fastened together side by side and were 60 feet long, this length being obtained by using two pieces, one 34 and the other 26 feet long. A tongue-and-groove joint was made by spiking to each piece of piling three 3×4-inch sticks, two on one side and one on the other, thus making each piling unit 30 inches wide. To give additional water-tightness to the cofferdam a large amount of filling was banked around the outside.

DOUBLE-WALL TYPE.—This is a form but little used since it offers but slight advantages over the single-wall type and is considerably more expensive. It is more easily made water-tight than the single-wall form, but on the other hand, it is very little stronger because strength is almost entirely dependent on the amount of internal bracing used. Where strength must be obtained without the use of bracing the type described in Art. 69 should be used.

The cofferdams for one of the piers of the Chattahoochee River Viaduct had an inside framework, 39 feet long by 15 feet wide, which was composed of horizontal frames of 6×8-inch pine timber braced with one set of longitudinal and two sets of transverse timbers. These frames were spaced from 2 feet center to center on the bottom to 3 feet centers at the top and were held in place by vertical posts between them, the total height of the framework being 9 feet. The outside frames were sufficiently large for a 4-foot thickness of puddle and were connected to the inside frames by braces and rods. The framing was partly built on shore, launched, floated to place and there completed.

The bottom of the river had a seamy ledge covered with a layer of sand varying in depth from 6 inches to 3 feet. As soon as the framework was sunk two rows of sheet-piling, each row consisting of a double thickness of 2-inch pine plank,

were driven, care being taken to break joints. The bottom of the puddle chamber was then covered with two layers of sacks loosely filled with sand, after which the remainder of the chamber was filled with clay puddle. Considerable trouble was caused by water coming up in the cofferdam through the seamy ledge and this leakage was stopped only after a 2-foot layer of concrete was deposited through the water and allowed to harden before pumping out the water.

ART. 69. SHEET-PILING SUPPORTED BY CRIBS

For cofferdams which rest on hard bottom and are too large to employ internal bracing economically, a series of cribs, laid up log-house fashion, are used to hold the sheet-piling in place. Each crib unit is made as long as can be conveniently handled and as wide as is necessary to develop the required stability. Rough logs are generally used although in some cases they may be squared, but the latter offer only a slight advantage over the former. In building these cribs the bottom courses are usually started on land and the crib is built to a height sufficient to permit the top part being well out of water when it is first launched; after this it is launched, floated to place and completed. Where the stream is low at certain times of the year the cribs may sometimes be built in place. The bottom of each crib should be shaped to fit the rock bottom, and if a few feet of sand or other material overlies the bedrock this should be dredged out before placing the cribs. A part of the bottom of the crib is usually floored to permit placing stones so as to sink it.

After all the cribs are sunk the remainder of the space inside of them may be filled with stones or earth. The latter material possesses the advantage of not only giving the cribs great stability but also to secure water-tightness. After the cribs are placed sheet piling is driven around the outside and banked with earth. This type of cofferdam is very widely used in building dams for hydro-electric plants.

Fig. 69*a* shows a view of the cofferdam employed in the con-

struction of a dam for the Connecticut River Power Co., near Vernon, Vt. The width varied with the height of the cofferdam; for the upstream one the maximum width was 35 feet, while the maximum height was 42 feet, or 16 feet above normal water-level. The structure was of the rock-filled type made of round logs in 7-foot checks, with the face logs slabbed on the sides to give good bearing for the sheet-piling. The top of the cribs were floored with logs to serve as a walk and also as a protection against ice pressures. On the outside the cribs were sheet-piled with 3-inch spline-and-grooved spruce, and this in turn was banked with earth up to normal water-level.

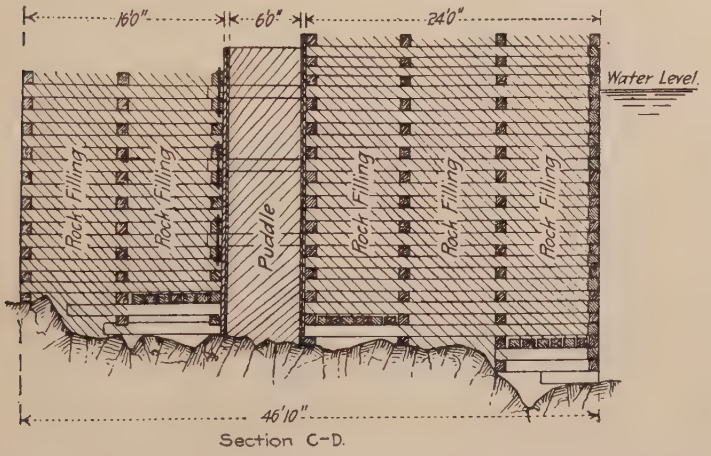


FIG. 69*b*.—Typical Section of Crib Cofferdam. Niagara Power Plant, Electrical Development Company of Ontario.

The cofferdams for the Niagara Power Plant of the Electric Development Co. of Ontario furnish an example of exceedingly strong and rigid cofferdams placed under the most trying conditions. In some places the current had a velocity as high as 17 feet per second which made it difficult to study the nature of the bottom and the depth of water previously to placing the cofferdams.

The widest part of the cofferdam consisted of two lines of parallel, rock-filled timber cribs with a space between, sheet-piled and filled with puddle as shown in Fig. 69*b*. Both cribs

were built of squared timber with the outside wall of the outer crib laid solid. The width of the cribs varied to meet the variation in depth and the bottom of the cribs was made to fit the irregularities of the rock surface. In shallow water the cribs were built in place but elsewhere they were constructed in the river upstream, and by means of cables from the shore they were floated into place and were sunk by filling with rocks the wells which had bottoms. For further details of this interesting cofferdam the reader is referred to Engineering News, vol. 54, page 561, Nov. 30, 1905.

ART. 70. STEEL SHEET-PILE COFFERDAMS

The advantages which steel sheet-piling possesses over the wooden type are discussed in Art. 60. On account of these advantages steel-piling is being used more and more in cofferdam work. The details of the structures differ but little from those using timber sheet-piling, the main difference being that the steel type, on account of the greater strength and positive interlock of the piling, requires less bracing.

Fig. 70*a* indicates a good example of a steel sheet-pile cofferdam with guide piles. In the illustration the guide piles and the outer course of wales are not shown, however. The bottom at the site of the pier consisted of hard-pan to an unknown depth covered with about 6 inches of mud. The depth of water was about 9 feet at mean tide, which had a rise and fall of about 6 feet. [1] "Round wooden piles were driven 8 feet apart enclosing the site of the 83×15-foot cofferdam; 6×12-in inside waling pieces were bolted to them above high water.

"Spacing blocks 4 inches thick and 12×12-inch inside wales were bolted to the outside wales, forming guides, between which were driven a single row of Lackawanna 12-inch, 40-pound steel sheet piles 35 feet long. These were all assembled together before driving . . . and then driven . . . by one McKiernan-Terry steam-hammer weighing 5000 pounds and making about 225 strokes per minute. It was handled by the

[1] Engineering Record, vol. 67, page 268, March 8, 1913.

boom of a floating derrick and went round and round the cofferdam, driving each pile a foot or two at a time until the work was completed. The driving was very hard, many boulders being encountered, some of which were displaced and others broken by the piles. When they could be neither displaced nor broken, driving on the piles that encountered them was discontinued, and adjacent piles were driven down to subgrade about 6 inches below the bottom of the footing.

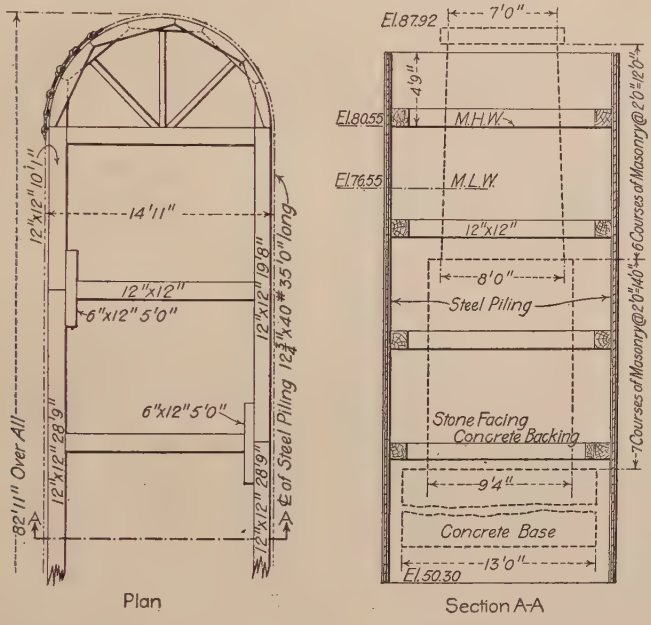


FIG. 70a.—Cofferdam for Highway Bridge Piers in Passaic River, at Bridge St., Newark, N. J.

"As the bottom was excavated inside the cofferdam, some of the boulders which obstructed the sheet piles were left in position and the sides of the excavation below them were closed as well as possible with bags of cement. The cofferdam resisted a pressure head of about 28 feet with very little leakage through the pile joints, which were packed with oakum. . . . The long sides of the cofferdam are braced with 12×12-inch horizontal transverse struts 9 feet 7 inches apart on centers,

in four tiers about 6 feet apart. At the rounded ends the inside waling pieces are made like arch centers of 12×12-inch double-scarf pieces, with radial braces to the middle of the adjacent cross-strut."

Some of the concrete piers for a bridge across the Illinois River at Peoria, Ill., were founded on bedrock 20 feet below the bottom of the river, where the depth of water was approximately 20 feet. To build these piers, cofferdams of steel sheet-piling on frames were used. By means of an orange-peel bucket the material of the river bottom was first dredged down to a layer of slate and soapstone, about 3 feet thick, which overlaid the rock. The excavation was made over a large area so that the material overlying the slate would stand at its natural slope and still leave an area on the slate of sufficient size for the cofferdams, one of which was 39 by 40 feet in plan.

[1]"The steel-piling forming the sides and ends of the cofferdam was braced across the latter with five longitudinal and six transverse rows of 12×12-inch timbers to hold it in place when the water had been drawn down in the cofferdam. These timbers were placed in nine horizontal layers, varying from $2\frac{1}{2}$ to 5 feet apart from the bottom to the top of the cofferdam. The horizontal layers were held apart by a vertical 12×12-inch timber at each intersection of the rows of braces. The timber crib formed by these braces and verticals was built in the water approximately over the site. The horizontal layer which would come at the level of the top of the slate and soapstone in the cofferdam was first assembled as a raft on which the verticals were erected and then the second horizontal layer was placed, sinking the crib thus formed to the water-level. The various horizontal layers were thus added in succession and when they had been completed the crib was towed over the site, sunk in position and anchored."

The steel-piling, of the Friestedt form, was driven around this framework through the slate and soapstone to rock, after which the material which had been previously dredged was backfilled around the cofferdam up to low water-level. After

[1] Engineering Record, vol. 55, page 247, March 2, 1907.

pumping out the cofferdam the layer of slate and soapstone was removed and the pier built.

Among the deepest cofferdams that have ever been placed are those used in founding the piers of the Tunkhannock Viaduct of the Delaware, Lackawanna & Western Railroad. These were land cofferdams and had a maximum depth of nearly 100 feet, with a depth of 65 feet below ground water-level. In principle they closely resemble the method used in placing piers for buildings as described in Art. 124, and differ from the regular caisson since excavation took place simultaneously with the driving of the sheet-piling, and since the lower part of the sheet-piling served as a form for the pier footing.

[1]"The cofferdam for pier 4 is typical of those of piers 3, 5, 6, 7 and 8 and was commenced by assembling on the surface of the ground a 43×49-foot rectangle made of 12×12-inch horizontal timbers spliced together to form one course of inner wales. Vertical posts were set up on this course and supported a second similar course about 16 feet above it, and two corresponding courses of exterior wales were erected outside of these and about 6 inches in the clear from them."

Lackawanna steel sheet-pile units 30 feet long were then placed between the outer and inner wales and driven by a steam-hammer going round and round the cofferdam driving each pile unit 2 or 3 feet at a time. As the piling was driven the interior was excavated and the cofferdam braced with successive tiers of 12×12-inch longitudinal and transverse struts.

After driving this set of piling to its full length an exterior row, concentric with the inner row and 4 feet 8 inches beyond the same, was assembled and first driven to a penetration of about 12 to 15 feet. The space between the two rows was then excavated and at the same time the inner row was also driven, the upper tiers of bracing of the latter being transferred to the bottom and new sets of bracing furnished to the outer piling. In this way, by driving both outer and inner rows to their required positions, the excavation was carried to rock.

[1] Engineering Record, vol. 67, page 485, May 3, 1913.

The advantage of two rows of piling was in the easier driving thereby obtained. The lower part of the excavation was completely filled with concrete, the steel-piling serving as a form;

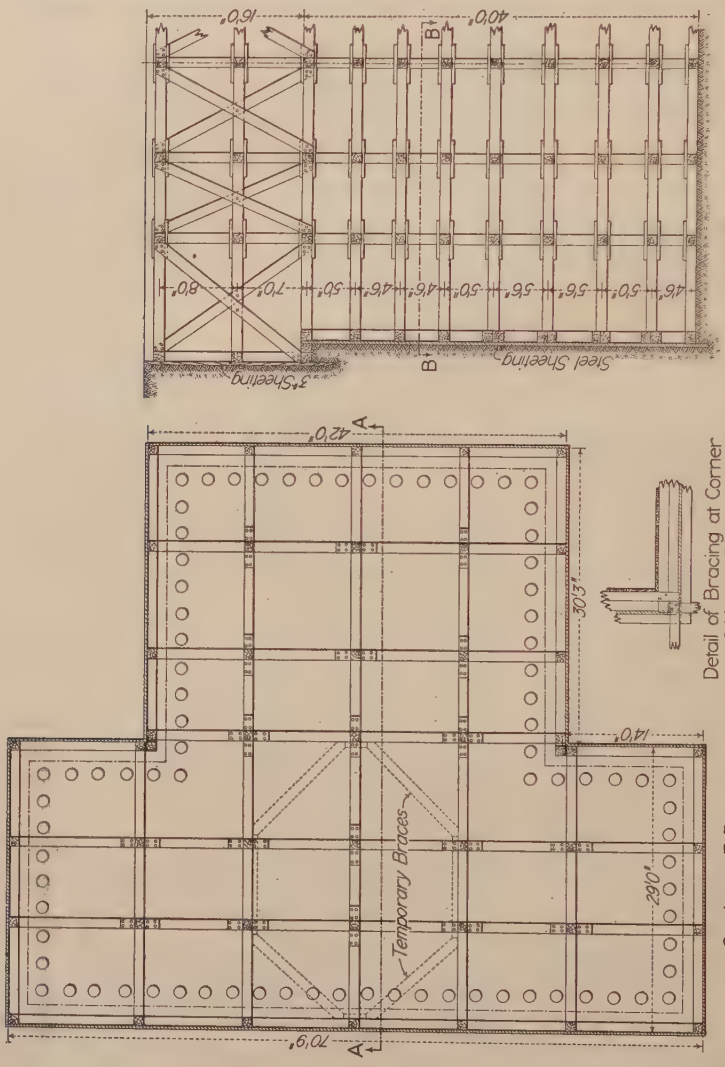


FIG. 70*b*.—Large Cofferdam for Abutment of Union Pacific Railroad Bridge over Kaw River, at Kansas City, Kansas.

the surface of the piling was protected from the concrete by tarred paper, thus permitting the piling to be with-drawn later.

Fig. 70*b* illustrates a somewhat similar type of cofferdam used

in the reconstruction of the Union Pacific Railroad bridge at Kansas City. The upper tier of sheet-piling was of wood. The details of the bracing are clearly shown in the illustration.

Fig. 70*c* is a half-tone showing the details of the bracing used for the steel sheet-pile cofferdam at the Loomis St. tunnel, Chicago. The cofferdam was 75 by 53 feet in plan and the maximum head of water on it was about 53 feet. The bracing consisted of 12×12-inch timbers, spaced 8 feet apart horizontally and 4 feet vertically.

Few examples exist of the type of cofferdam consisting of steel sheet-piling on cribs. The reason for this lies in the fact that almost all the crib and sheet-pile cofferdams have been built in localities where timber is abundant and for this reason sheet-piling of wood is cheaper than that of steel.

Art. 71. Self-Supporting Steel Sheet-Pile Cofferdams

There is a type of cofferdam using steel sheet-piling which has almost no parallel in the wooden sheet-pile cofferdam; this is the cofferdam without horizontal guides or bracing. Two reasons may be given for this fact: First, with the positive form of interlock which most forms of steel sheet-piling possess, sufficient guidance is furnished by the interlock to do away with the necessity of horizontal guides; and second, the higher strength lessens the amount of bracing necessary.

The two most notable examples of this type of cofferdam are those used for the United States Government lock at Black Rock Harbor, Buffalo, N. Y., and for raising the United States Battleship "Maine" in Havana Harbor, Cuba. Both of these structures rank high as daring pieces of cofferdam work, the former on account of its great size and the latter because of its great height.

The Black Rock cofferdam was built to permit the construction of a ship lock, and was rectangular in plan, 260 by 947 feet over all as shown in Fig. 71*a*. The depth of water at the site varied from 2 to 15 feet, averaging about 8 feet, while the solid rock on which the lock was built was about 40 feet below mean water-level. As shown in Fig. 71*b* the sides of the cofferdam

were made of two walls of steel sheet-piling, the space between the two walls being divided into pockets 30 feet square by transverse walls of the same piling as that used for the main walls, which served to connect the latter. A horizontal 15-inch, 40-pound channel was bolted to the tops of the piles of the inner wall and a similar channel was bolted at an inclination across the transverse walls as shown in Fig. 71*c*.

The piling was driven to rock and at first wooden guide piles and wales were used to maintain the alignment of the steel sheeting, but eventually these guides were dispensed with, the

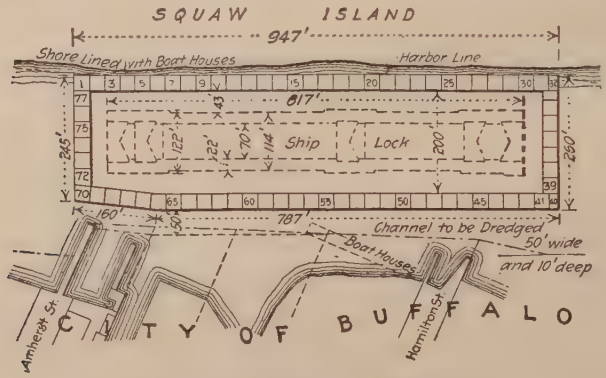


FIG. 71*a*.—Plan of Black Rock Cofferdam.

only ones used being 10×30-foot floating forms having one edge in the plane of the sheeting. The fine alignment attained by this simple method may be seen in Fig. 71*b*. After driving the piling the pockets were filled with clay and to further strengthen the structure, as the inside was excavated, a bank of earth 25 feet high was maintained on the inside as shown in Fig. 71*c*. But in spite of this bank of earth the material in the pockets caused the inside wall to bulge badly between the cross walls in both a horizontal and vertical direction.

It is instructive to observe the plans of different pockets of the cofferdam, and the curvature of vertical sections after the steel sheet-piling adjusted itself to the pressure of the clay filling by developing tension in the interlock. Fig. 71*d* gives the

FIG. 71*b*.—Cofferdam for the U. S. Government Ship Lock at Black Harbor. Northeast Corner

(*Facing p.* 222.)

FIG. 71c.—Cofferdam at Black Rock Harbor, Looking North from Southwest Corner. Shows bulging of pockets.

results of a careful survey of pocket No. 35 in which the maximum bulging of sides occurred. It should be noted how short a distance the bulging extended below the sand and gravel bank which was allowed to remain inside of the cofferdam. The diagram also shows vertical sections at the middle of pockets Nos. 30, 52, and 75, the relative location of the pockets being indicated in Fig. 71*a*. See also the half-tone view, Fig. 71*c*.

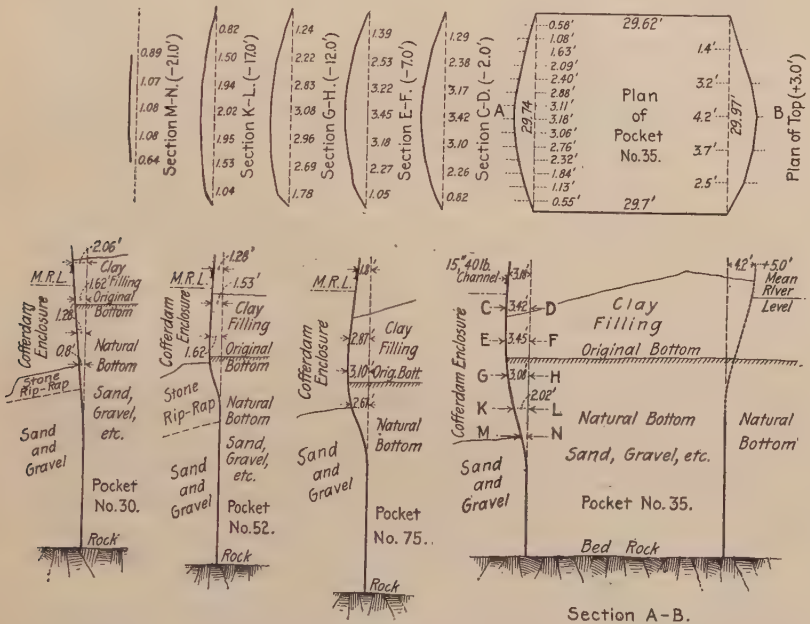


FIG. 71*d*.—Diagram Showing Deformation of Steel Sheet Piling.

In all 6589 tons of steel sheet-piling were used in this cofferdam, there being 6870 linear feet of piling wall from 45 to 50 feet high, which makes this the largest piece of cofferdam sheet-piling work on record to 1914. The price paid the contractor for building the cofferdam, which included furnishing all material, pumping out, and maintaining the same, was $408 830. The type of piling used was that known as the Lackawanna, and which had a web thickness of $\frac{1}{2}$ inch and weighed 40 pounds per linear foot.

The cofferdam for raising the "Maine" represents a special type of steel cofferdam, very large and strong. [1] "The problem was to surround the wreck of the vessel, lying in about 29 to 37 feet of water, with a cofferdam, which when unwatered would be tight enough to prevent leakage, strong enough to resist outside water and mud pressures, and a protection that would assure safety during the work. The cofferdam should be self-sustaining, if possible. Bracing by struts across its interior to resist the water and mud pressures might be difficult to install and would interfere with the operation of removal. The borings indicated bad conditions for foundations. The building of a cofferdam without internal bracing, which would withstand pressures from a head of 37 feet of water and practically 21 to 23 feet of mud, was an unprecedented task.

"The cofferdam should be not only self-sustaining and safe against the pressures to which it was to be exposed, but it should also be capable of complete removal after it had served its purpose. It should be able to support more or less superimposed loads, for working platforms had to be built upon it. The work of unwatering the area enclosed had to be carried on from the top of the cofferdam; and afterward, men and materials had to be transferred from there to the interior, for work upon the wreck. . . . The cofferdam decided upon consisted of 20 equal cylinders, 50 feet in diameter, and composed of steel-piling 75 feet long. . . ." A plan is shown in Fig. 71*e*.

"The length of the major axis of the cofferdam was practically 399 feet, and of the minor axis 219 feet, leaving a 20-foot clearance at the submerged bow of the ship and a 14-foot clearance at the stern, with 45 feet at the side cylinders. Such clearance was necessary to avoid portions of the wreck which had been blown beyond the position occupied by the hull.

"The units of the cofferdam were made cylindrical for the reason that the extremely high pressures, which would be exerted by the mud filling, would act radially and uniformly on each pile, straining each joint to the same amount at equal depths,

[1] Bulletin No. 102, Lackawanna Steel Co., Buffalo, N.Y.

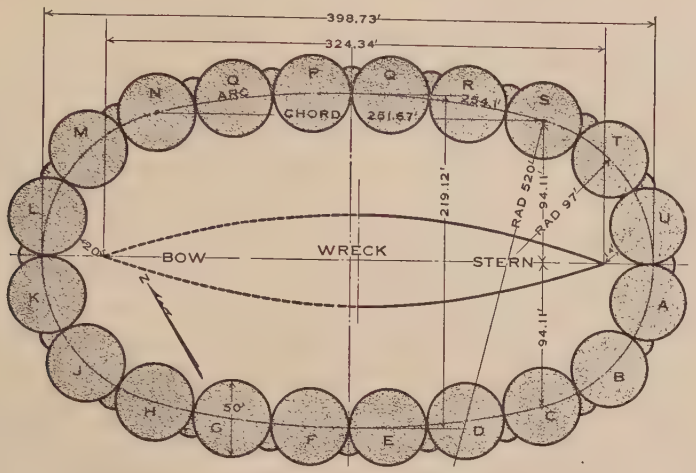


FIG. 71*e*.—Plan of Cofferdam for Raising the "Maine."

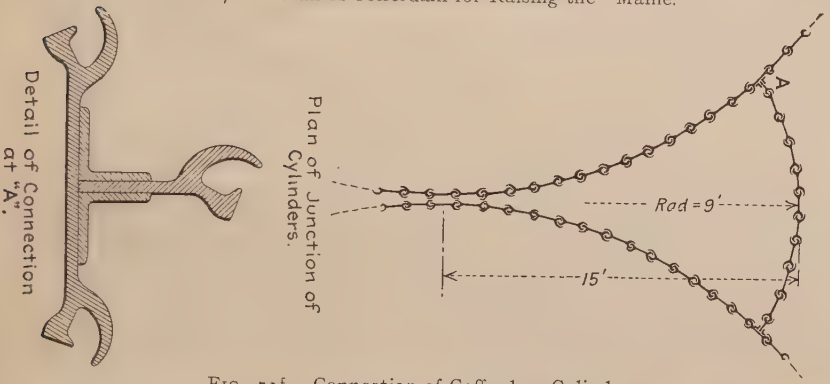


FIG. 71*f*.—Connection of Cofferdam Cylinders.

FIG. 71*g*.—Filling Clay into Cylinder *A*. Part of *B* in Foreground.

and in the entire cofferdam cylinders would deform least from play in the piling interlocks."

The cylinders were driven tangent to one another and to insure their stability and prevent leakage of water through them when the cofferdam was pumped out they were filled to the top with clayey material that was dredged from the bottom of the harbor. A curved diaphragm of steel-piling, as shown in Fig. 71*f*, was driven to connect adjacent cylinders, and the space between this arc and the outer surfaces of the large cylinders was likewise filled with dredged material.

The piling used was the Lackawanna section, weighing 35 pounds per linear foot, and had a web $\frac{1}{2}$ inch thick. The piles were driven so that their tops were 2 or 3 feet above normal water-level (Fig. 71*g*) and the 75-foot length of piling, which penetrated the harbor bottom to a distance of approximately 35 feet, was made of two lengths spliced together with channels.

ART. 72. CRIB COFFERDAMS

Where the cofferdam is to rest on bedrock which is approximately smooth and level, a crib cofferdam, formed with one or two walls of squared horizontal timbers laid closely, may be used in place of the sheet-pile cofferdam. Where the single-wall type is used it is ordinarily made an integral and permanent part of the pier, and as such is not a cofferdam, but a caisson. For a description of this type see Art. 83.

In his book on Sub-aqueous Foundations, FOWLER describes a double-wall crib cofferdam used by the C. B. & Q. R. R., which was made from 2×8-inch and 2×10-inch fence boards laid flat. The two walls were thoroughly tied together and the space between filled with puddle.

Fig. 72*a* shows a polygonal cofferdam of the crib type which was used for the center pier of the Arthur Kill Bridge. At the site of the cofferdam the depth of water at high tide was about 28 feet, with about 4 feet of mud and clay overlying bedrock. This mud and clay was dredged out previously to placing the cofferdam. The latter had twelve sides with walls 4 feet apart in the clear, and in this space puddle was dumped. All courses

of timber were thoroughly drift-bolted together and all joints caulked with cotton wicking. No internal bracing was used. Before pumping out the water a 4-foot layer of concrete was deposited all over the bottom and allowed to harden for a week.

The cofferdam for the new inlet tower of the St. Louis Waterworks was of the double-wall crib type, 38 by 76 feet in plan and 22 feet high. The walls were composed of horizontal 12×12-inch material and were 3 feet apart in the clear. The joints between all courses were carefully caulked. The cofferdam was braced transversely by three vertical rows of

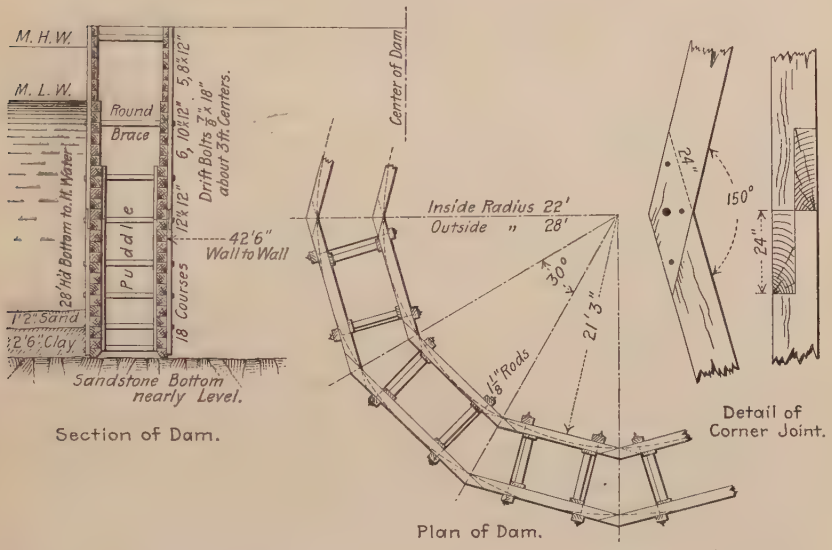


FIG. 72a.—Cofferdam for Pivot Pier of Arthur Kill Bridge.

horizontal 12×12-inch timbers spaced 4 feet apart vertically, and extending from outside wall to outside wall, thus tying the walls together as well as bracing the cofferdam. The ends were braced by similar horizontal 12×12-inch diagonal timbers, running at an angle of about 45 degrees from the center of the ends to the sides.

The river bottom was bedrock and the depth of water about 15 feet, the current having a velocity of from 6 to 8 miles an hour. The cofferdam was held in place by three triangular

cribs filled with rocks and sunk upstream from the cofferdam and tied to the latter by cables. The puddle chamber was partly filled with concrete in sacks and puddle placed on top. Sacks of clay were also banked around the outside.

Cofferdams are widely used as temporary adjuncts to open and pneumatic caissons, but as the details differ widely from the types described in this chapter and resemble closely the caissons themselves they will be described in the chapters dealing with such caissons.

ART. 73. MOVABLE COFFERDAMS

Unless it forms an obstruction to navigation only that part of the cofferdam above low water is sometimes removed. This is because the salvage value of the material is less than the cost of getting it out, except where steel sheet-piling is used.

Where the same size and style of cofferdam is to be used for a number of piers it will often prove advantageous to so construct a cofferdam that it can be used over and over again. In one type, that of the cofferdam on grillage, it is so easy to make its sides removable that it is universally done, even though they may not be used a second time.

A movable cofferdam consisting of sheet-piling supported by a crib was used in constructing the piers of the Falls-of-Schuylkill Bridge, of the Philadelphia & Reading Railroad. When in position the cofferdam was 62 feet long, 36 feet wide and 16 feet high. The cribs were 10 feet thick, making the inside dimensions 42×16 feet. The cofferdam was divided vertically through each short side into two parts of equal size and these were floated separately to the site, joined together and sunk. Each section had water-tight compartments to assist in floating and these were filled with water and stone, while other non-water-tight compartments were filled with stone, when it was desired to sink the sections. On reaching the rock bottom sheet-piling of jointed planks, 3 or 4 inches thick, was placed on the outside and spiked there. Puddle was then placed around the outside, after which the cofferdam was

pumped out. Two sets of horizontal bracing connecting the long sides were placed as the water was removed.

In placing cylinder piers for the Queen's Bridge, Melbourne, Australia, square movable cofferdams of the sheet-pile-on-frame type were used. One side opened outward as a door, thus permitting the cofferdam to be removed on completion of a pier.[1] "The dam was built on shore complete, and launched ready for immediate use on the site of a cylinder. The sheet-piling was vertical and consisted of 12×4-inch rough-sawn Oregon planks, supported by horizontal frames of 12×12-inch Oregon timber, spaced close together near the bottom of the river, to

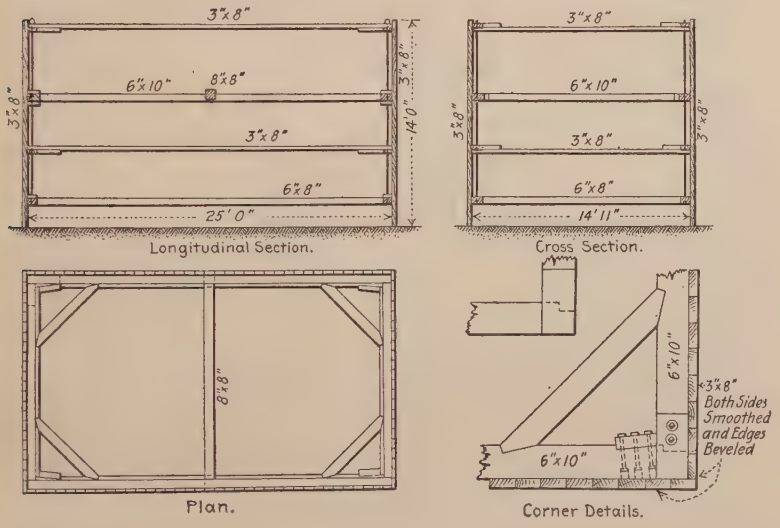


FIG. 73a.—Cofferdam Used on Key West Extension of Florida East Coast Railway.

carry the greater pressure of water. Up the four corners of the dam were 12×12-inch Oregon timbers, into which the frames were checked and by which they were kept to their proper spacing, and which formed supports for the door. Outside the sheet-piling, at the top and bottom frames there were outside wales, 12 by 6 inches (keeping the sheet-piling in place), bolted to the frames inside by 1-inch bolts, two to each wale, passing between two sheet piles." The sheet-piling was flush

[1] Engineering News, vol. 33, page 230, April 4, 1895.

with the bottom of the frame and extended a few feet above the top.

At the site of the piers there was about 3 feet of soft silt covering the rock. This silt was covered with puddle before placing the cofferdam. After sinking it by weighting, the sheet-piling was driven through the puddle and silt. On pumping out the cofferdam much of the silt ran into the interior and the clay took its place, thus sealing the structure. To remove the cofferdam the sheet-piling was first drawn up, the loading taken off, the door opened, and the cofferdam floated out. At

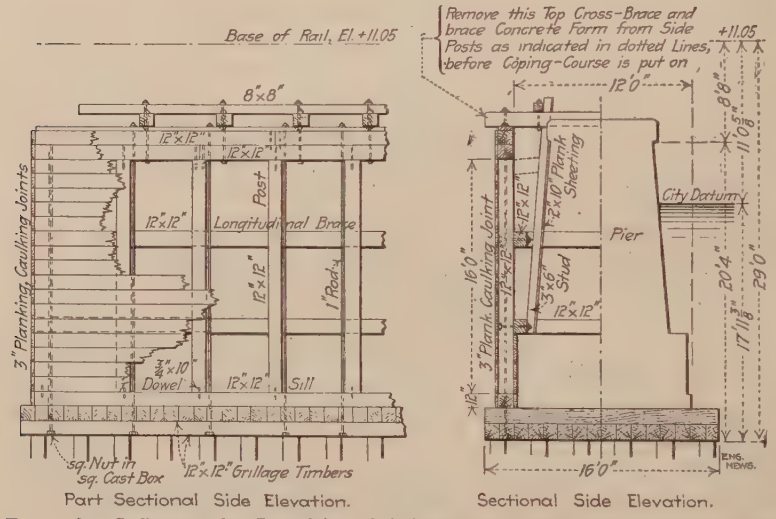


FIG. 73*b*.—Cofferdam for Rest Pier of Chicago and Northwestern Railway Lift Bridge at Kinzie Street, Chicago.

first tarpaulin was placed around the outside of the cofferdam but it was later found that this was unnecessary since the sheet-piling was water-tight without it.

Fig. 73*a* illustrates the form of a movable cofferdam used in constructing the piers of the Key West Extension of the Florida East Coast Railway, where the depth of water did not exceed 8 feet. The two sides and the two ends formed independent portable sections which were connected together by means of $1\frac{1}{4}$-inch vertical rods running down through the overlapping rangers at the corners of the cofferdam.

At the site of the piers sand overlaid the coral rock. Piles, for the foundation of the pier, were first driven until the tops were 2 feet below low water, after which the cofferdam was assembled on a barge, lifted from the same and set in place. The sand was then pumped out by a centrifugal pump, after which a 2-foot seal of concrete was placed over the whole bottom. After allowing this concrete to harden for seven days, the cofferdam was pumped out, forms placed and the pier built. On completion of the pier the rods were withdrawn, which allowed the sections to float free.

MOVABLE COFFERDAMS ON GRILLAGE.—On account of its convenience and ease of manipulation a movable cofferdam is almost universally employed where a timber grillage foundation on piles is used for the pier. The grillage and the cofferdam form an open box constructed on shore or on a barge or raft, launched, floated to the site, and sunk on the pile foundation by building the pier in the box. This type differs from the box caisson, described in Art. 80, since the sides of the former are not a permanent part of the pier. After the pier is built to above high-water level the cofferdam is removed, the sides being so fastened to the grillage that this can easily be done.

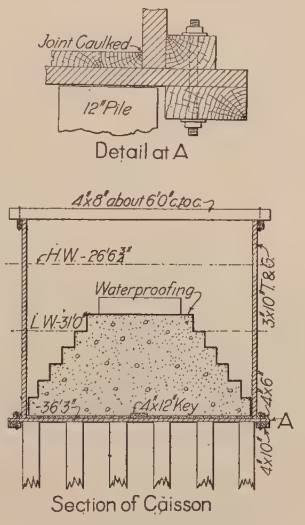


FIG. 73*c*.—Cofferdam with Removable Sides.

Fig. 73*b* shows the details of the movable cofferdam used for the 12×41½-foot pier of the Kinzie St. drawbridge, in Chicago. This cofferdam was connected to the grillage by 28 vertical 1-inch rods, 21½ feet long. To sink the structure, concrete forming the pier was placed in the same, the cofferdam itself serving as a form for the concrete up to an elevation shown in the drawing, and above this regular forms were used. On com-

pletion of the pier the rods were removed, which permitted the removal of the cofferdam from the grillage.

A very simple cofferdam on grillage was used in building the foundation piers of the Bellevue Hospital Boiler House, a section of which may be seen in Fig. 73*c*. The largest was approximately 14×52 feet in plan and 12½ feet high. The most interesting feature is the very thin grillage used, it being composed of two crossed courses of 2-inch tongue-and-grooved planks. It was desired to use a thickness which would give enough strength for launching and sinking stresses, and yet be sufficiently flexible so that a uniform bearing over the slightly irregular pile tops would be secured.

ART. 74. MISCELLANEOUS TYPES

The foregoing articles have dealt with what may be called standard types of cofferdams, but there have been many cofferdams constructed which are either a combination of any two standard types or which differ fundamentally from those which have been described. For instance in the cofferdam for the Dearborn St. bridge, Chicago, a double-wall sheet-pile cofferdam was used, the outer wall being composed of Wakefield sheet-piling and the inner wall of Friestedt steel sheet-piling, thus giving a composite wood and steel sheet-pile cofferdam.

Fig. 74*a* shows a form of cofferdam known as the A-frame type which is used on bedrock. This particular one was used on the New York Barge Canal and consisted of a series of bents, spaced 6 feet center to center. On these bents rested purlins, which in turn supported the sheathing of jointed and caulked 3-inch planking. As shown in the illustration, the structure was braced against sliding by having certain of the struts bear against concrete footings on the rock. This form of bearing can be made only when the rock is exposed at times. The maximum head of water supported was 18 feet.

Another cofferdam of the same type was used for some canal work at Keokuk, Iowa. Here it was necessary to construct the cofferdam without drawing off the water in the canal,

and hence it was built away from the site, brought to place, and sunk by weighting with iron rails. Water-tightness was promoted by covering the sheathing with canvas.

In constructing concrete wharves at Fort Mason, San Francisco, steel-cylinder cofferdams, 7 feet 7 inches in diameter and 50 feet long, were used in which to construct reinforced-concrete piers. These cylinders, weighing 17 tons each, were driven by a pile-driver through the bottom and into hard-pan, after which the water was bailed out, the mud removed, wooden

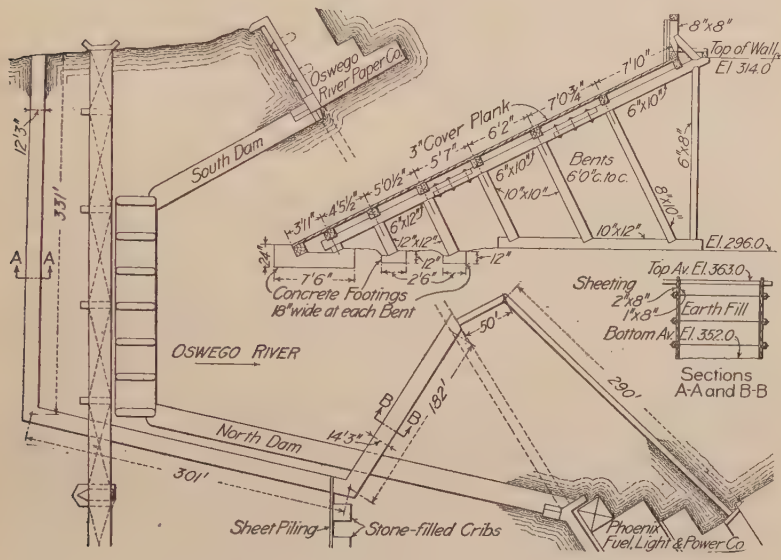


FIG. 74a.—A-Frame Cofferdam Used on New York State Barge Canal.

forms placed and the 4-foot reinforced-concrete piers with enlarged bases, $6\frac{1}{2}$ feet in diameter, cast. After the concrete had set the cylinders were pulled by the pile-driver, the required pull being about 50 tons.

In vol. 26 of Revue Technique, the proposed design for a cofferdam of ice to close the entrance to the outer basin at the Port of La Rochelle is described. The freezing was to be done with refrigerating machines, sheathings of non-conducting material being placed between the water to be frozen and that

to be left unfrozen. It was estimated that such a cofferdam would cost less than any ordinary form.

Another proposed type described in the same article was of reinforced concrete. It was designed with inclined sides, the width of the base being 19.7 feet, the width of the top 4.9 feet, and the thickness of the walls about 10 inches. The walls were to be well braced by struts. The structure was to be built away from the site, floated to place and sunk by filling with clay. It was estimated that it would have sufficient stability to withstand a head of 24.6 feet of water.

ART. 75. PUDDLE AND LEAKAGE

It is seldom attempted to construct a water-tight cofferdam, for the cost of such is usually prohibitive. The greatest trouble from leakage occurs in the case of cofferdams which rest on rock, for here it is almost impossible to prevent the water from running in between the bottom of the cofferdam and the rock. But if the structure rests on clay and the sheet-piling is driven well down there will be but a slight amount of leakage at this point. Where the leakage occurs through seams in the rock it may be stopped by filling the seams with grout pumped in through pipes about 4 inches in diameter. Another method is to dump clay, sand, ashes, etc., all around the cofferdam with a view of shutting off the water-supply of the crevices.

When the leakage is due to irregularities in the rock surface, concrete in bags may be placed on the bottom, or water-logged oat straw may be sunk by mixing with ashes or covered with a wire net loaded with sand and clay, after which the rest of the puddle filling may be placed. Another method of preventing leakage on a rock bottom is to use canvas as noted in previous articles.

Leaks often develop in double-wall cofferdams by the filling between the walls not compacting well, or settling after being placed and leaving openings beneath cross braces. To compact this filling piles are sometimes driven into it or stock ramming may be resorted to. The latter consists of forcing clay cylinders through pipes into the filling.

The ideal puddle for a cofferdam is a mixture of clay and sand or clay and gravel. The sand or clay alone is almost worthless, the sand because of its permeability and the clay on account of its tendency to allow a leak once started to enlarge rapidly through the clay arching over the leak, instead of falling into and stopping the same. The ideal mixture obtains when there is just enough coarse material in it to reduce the cohesion of the mass sufficiently to prevent this arching action.

ART. 76. DESIGN OF COFFERDAMS

Like most structures used in foundations, a purely theoretical design of cofferdams leads to unsatisfactory results. For some types it is a simple matter to design the structure to resist the hydrostatic pressure, but to design it properly to resist safely the pressure of the earth filling, or of freshets, ice, or floating logs, requires much experience.

Earth cofferdams usually fail by the water seeping through and enlarging a channel until a washout takes place. For this reason such cofferdams should be carefully watched to detect small leaks that they may be checked quickly after starting. In general, if the cofferdam is madeo f a good mixture of clay and sand, has a width of at least 3 feet at the top, which is well above high water, and has sides inclined at the natural slope of the material, the cofferdam will be safe.

In the single-wall sheet-pile cofferdam with guide piles, if the wales are at the top and bottom, the sheet piles may be assumed to act as simple beams, with a load per vertical foot varying uniformly from zero at the water surface to a value of wd pounds per square foot at the surface of the earth, where w is the weight in pounds of a cubic foot of water and d the depth of the water in feet. For a discussion of the design of sheet-piling see Art. 63. The wales take the reactions of the sheet-piling and transfer them as supported, partially continuous, or continuous beams, to the guide piles. Conservative engineers usually design the wales as simple or supported beams. If no bracing is used the guide piles should be designed as cantilever beams with loads coming from the waling pieces. The maxi-

mum moment will occur at or below the mud line. If firmly braced at the top by struts extending across the cofferdam, it will be best to design the guide piles as simple beams.

Each wall of the double-wall sheet-pile cofferdam with guide piles may be designed somewhat in accordance with the above outline. The outer wall will be subjected to water pressure from the outside and earth pressure from the inside. Experience shows that usually the pressure from the puddle will, for equal heads, be larger than the pressure from the water. This will cause a stress in the tie rods connecting the two walls. The inner wall must be designed to resist the forces due to the puddle filling.

In the design of a cofferdam composed of sheet-piling on frames and the corresponding bracing, the hydrostatic pressure is the only force to be considered. The sheet-piling acts as a simple beam between horizontal rangers, and the latter act as beams between bracing struts.

The sheet-pile-on-crib cofferdam must be designed so that the cribs will not overturn or slide. To be safe against sliding the weight of the cribs and filling per linear foot of length, multiplied by the coefficient of friction between the crib and rock, must be greater than $wd^2/2$, where the terms have the same meaning as those previously given. To be safe against overturning the weight per linear foot of length, including filling, multiplied by one-half the width, must be greater than $wd^3/6$.

Before attempting to design a cofferdam the literature on the subject should be carefully read, for as stated in the first part of the article no purely theoretical design will result in a thoroughly satisfactory structure. In the preceding articles, standard types and standard methods of construction are described and a careful reading of this material will help the inexperienced engineer. For more detailed information the reader is referred to the carefully selected list of references in Chap. XIX.

ART. 77. COST OF COFFERDAMS

Little value attends the mere statement of cost of engineering works unless all the conditions are fully described (see Engineer-

ing News, vol. 70, page 1305, Dec. 25, 1913). For this reason only a few figures will be given here. In Art. 71 the cost to the United States of the steel sheet-pile cofferdam at Black Rock Harbor is given.

Thomas P. Roberts writes[1] that for large cofferdams constructed in the rivers near Pittsburgh, Pa., the cost per linear foot of cofferdam will vary from $8 to $10. These cofferdams are of the double-wall type, from 10 to 12 feet wide and from 14 to 16 feet high. Two-inch hemlock sheet-piling is used.

In constructing some piers for a bridge over Paint Creek, near Chillicothe, Ohio, where the water was from 3 to 6 feet deep, a single-wall steel sheet-pile cofferdam, 16 by 62 feet in plan, was used. The piling was 16 feet long and was driven into the gravel bottom until the top of the same was 2 feet above water-level. The bracing consisted of two horizontal wales at the top, running longitudinally and cross-braced with struts. The first cost of the sheet-piling was about $1822, and as the same piling was used for five cofferdams, the cost per cofferdam was about $364. The cost of placing two of the cofferdams averaged $94, while the cost of removing the piling per cofferdam was $47, thus making the total cost of each cofferdam about $505.

In 1886, near the same site, some cofferdams of the double-wall type were built with wooden sheet-piling on guide piles. For the river piers the cofferdams were 22 by 45 feet inside and 35 by 58 feet outside. The guide piles were about 8 feet apart. The wales were 3 inches thick and the sheet-piling was made of 2-inch planking. The bid for the construction of these two cofferdams averaged about $569, the unit prices being as follows: timber $24 per 1000 feet B. M., piles 30 cents per linear foot, iron bolts 5 cents per pound, and earth filling in cofferdam 30 cents per cubic yard. At the time the steel cofferdams were built (about 1905) the cost of the double-wall cofferdams would probably have been between 30 and 40 percent greater than in 1886. These figures show the considerable economy of cofferdams in which steel instead of timber sheet-piling is employed.

[1] Engineering News, vol. 54, page 138, Aug. 10, 1905.

ART. 78. CHOICE OF TYPE

The best type to use in any particular case is that one which fulfills all the required functions at a minimum cost. Where the depth of water is not great, and the danger of overflow and washing away does not occur, the simple earth cofferdam will prove the cheapest and most satisfactory, especially if the site of the permanent foundation must be excavated to some depth; for in this case the excavated material may be used to form the cofferdam. Where the depth of water is considerable the width of the cofferdam becomes so great that this type is not economical.

Where the bottom can be penetrated with piles the sheet-pile cofferdam with guide piles is a very satisfactory type. For high heads the double-wall cofferdam will be used. This form approximates somewhat the earth cofferdam, but possesses the advantage over it that less earth is required, and it is also a stronger structure and more nearly water-tight. The single-wall type obstructs the water-way less than does that with double walls, but it has less strength. If bracing can be used on the inside the latter can usually be made sufficiently strong to withstand any forces that are likely to come upon it.

Where the bottom is composed of rock a sheet-pile cofferdam on a frame or cribs will be used. Frames are used where the cofferdam can be braced across by struts, but where the structure is too large for such bracing, cribs are necessary. The crib cofferdam may be said to have gone out of use, the open caisson having taken its place. Where the cofferdam is not large and the same size is to be used a number of times some form of movable structure should be adopted.

As to whether wooden or steel-piling should be used in any particular case becomes simply a question of the relative cost of the two types. In general, the steel-piling will be used in and near cities or where the work is in close proximity to a railroad, while the wooden sheet-piling will be cheaper near centers of timber supplies. Steel-piling will also show more economy the greater the depth of cofferdam.

ting edges and along the sides the frictional resistance is considerably decreased. Another advantage is that it tends to wash the material toward the interior of the caisson, where it can be picked up by the dredging buckets, which have previously made a hole in the center.

It is possible to drive caissons only when they are small and even then only light blows with the hammer may safely be given. Pulling the caisson down may sometimes be employed to advantage, if it is possible to drive piles around the outside and attach tackle to them and to the sides of the caisson.

CHAPTER VIII

PNEUMATIC CAISSONS FOR BRIDGES

Art. 92. The Pneumatic Process

The use of the plenum pneumatic process for founding deep piers is a good example of the application of scientific principles to foundation work. A pneumatic caisson may be defined as a structure, open at the bottom and closed at the top—in other words, an inverted box—in which compressed air is utilized to keep the water and mud from coming into the box, and which forms an integral part of the foundation.

The caisson, which is usually not over 6 feet high in the working chamber, is surmounted by a crib and cofferdam, the former, with the exception of one or more vertical wells, called shafts, being filled with concrete as the caisson sinks. This concreting, together with the excavating done in the working chamber, as the interior of the caisson is called, effects the sinking of the latter.

The working chamber must be practically air- and water-tight, and yet there must be an opening for men to enter and leave the chamber, as well as an inlet and outlet for materials. These openings are provided by vertical shafts and air-locks. The shafts, which extend from the roof of the caisson to a point well above the top of the crib and the level of the water outside, are usually of a circular or oval section and from $2\frac{1}{2}$ to 4 feet in maximum diameter. In the shafts, at the bottom, top, or between these two points, are placed the air-locks, they being air-tight chambers, often simply a part of the shaft itself, fitted with two doors, one of which leads to the working chamber and the other to the open air.

The most pronounced advantage of the pneumatic caisson as compared with the open-caisson process lies in the fact that the

engineer has more control over the work, having a better opportunity to sink the caisson vertically, to remove large boulders, sunken logs, etc., from under the cutting edge; the foundation bed can be properly prepared and personally inspected; and lastly, the concrete filling of the working chamber is deposited in air, thus giving a superior foundation. Another point, which is sometimes of great importance in placing foundations for buildings, is that the soil about the caisson is not so liable to be disturbed when the pneumatic process is used. The one disadvantage of this process is that the men have to work under an air pressure which is sufficient to balance the pressure of the surrounding water in addition to atmospheric pressure, or practically the full hydrostatic head from the cutting edge to the water surface.

For depths from about 30 to 110 feet this type of caisson is extensively employed. For depths less than 30 feet the cofferdam process is usually, but not always, a more economical method of placing the foundation while for depths greater than about 110 feet, corresponding to a pressure of over three atmospheres above the normal, the open-caisson method must be employed, since men cannot work advantageously under such high pressures.

At the St. Louis Municipal bridge men worked at a maximum immersion of over 113 feet, the maximum gage pressure being 50 pounds, which is probably the world record for bridge caissons, with the possible exception of a bridge caisson in Denmark, where it is reported (Eng. News, vol. 26, page 467, Nov. 14, 1891) that a working depth of 115 feet was reached. Among other notable examples of deep immersions are the St. Louis arch bridge caissons, 109.7 feet; the Memphis bridge caissons, 106.4 feet; the Williamsburg bridge (New York) caissons, 107.5; and the Broadway bridge (Portland, Ore.) caissons, 101 feet. The elevation of the bottom of the deepest caisson (No. 4) of the St. Louis Municipal bridge is 2.1 feet below the bottom of the east abutment caisson, or the deepest one of the St. Louis arch bridge.

The caisson used in sinking a mine shaft near Deerwood,

Minn., was sunk to a depth of 123 feet below ground-water level and 130 feet below the ground surface. The maximum pressure used was 52 pounds per square inch, a higher value than has probably ever been used in bridge or building caissons.

As first applied the pneumatic caisson process was a very simple affair, the caisson consisting of a cast-iron cylinder, called a pneumatic pile, which formed both the working chamber and a section of the pier. The first used in this country were sunk in 1852 in the Pedee River, North Carolina.

The St. Louis arch bridge was the first in this country to be founded on large pneumatic caissons, its east abutment caisson, which had a maximum immersion of 109 feet $8\frac{1}{2}$ inches, being sunk in 1870. The second bridge in this country to be founded on large pneumatic caissons was the great Brooklyn suspension bridge, which, in its New York tower caisson, sunk in 1871, has the largest pneumatic caisson ever placed for a bridge foundation. It was 102 by 172 feet in plan and was sunk to a depth of 78 feet below high-water level.

The pneumatic caisson process has been widely used in America and on the European continent. As a class, English engineers have apparently shown some aversion to it, and in many cases, where it seems to have been the preferable structure on account of the presence of boulders and logs, the open-caisson process was used. American engineers have developed the wooden caisson to a high state of perfection, but at present (1914) owing to the high price of timber the tendency is toward the use of more reinforced concrete and less timber. In Europe the metallic form of pneumatic caisson has been extensively used.

To give some indication of the progress made in the science and art of foundation construction it is interesting to note that the cost per cubic yard of the substructure of the Municipal bridge at St. Louis is only 29.6 percent of the corresponding cost of that of the St. Louis arch bridge, which is located about a mile above it, and 50.8 percent of that of the Memphis bridge. The substructures of these three bridges were completed in 1911, 1871 and 1891 respectively. In this comparison the

approaches are excluded. As previously noted, the three bridges have deep foundations.

The contract price for the caisson work of the Municipal bridge was $27.00 per cubic yard below the cutting edge, and $12.90 per cubic yard from the cutting edge to the top of the crib.

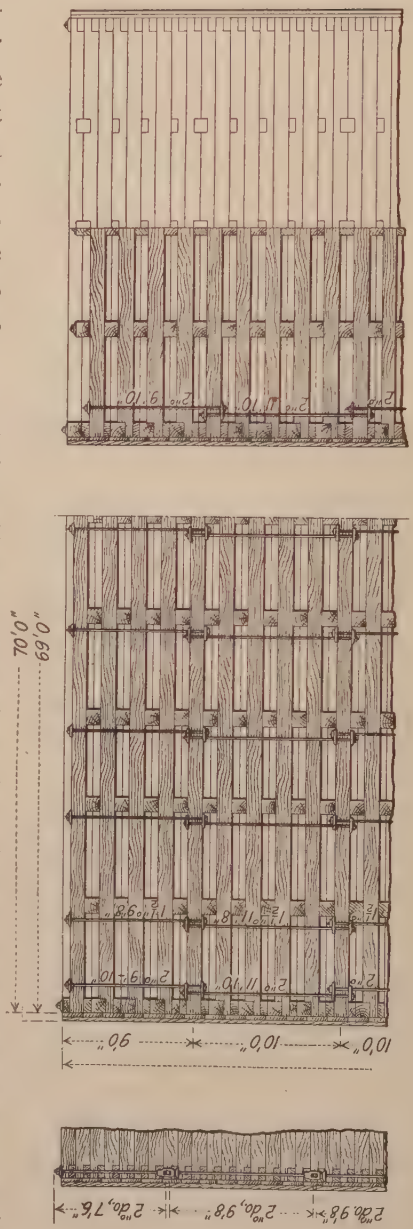
ART. 93. CAISSON ROOF CONSTRUCTION

TIMBER ROOFS.—The design of the roof has always been largely a question of judgment as it is almost impossible to analyze the stresses. The tendency of roof construction has been constantly to decrease the thickness of the timber roof and consequently its cost. When concrete superseded stone masonry as a filling for the crib a considerable decrease in the thickness of the roof was made possible on account of the strength of the concrete. A more generous use of bulkheads and the arrangement of the bracing above and below the deck to act as trusses also aided in securing a thinner roof. At present many caissons do away with a permanent timber roof almost entirely by reinforcing the concrete filling of the crib.

The roof is usually made with layers of 12×12-inch timbers, sheathed on the lower side with 2- or 3-inch planks. Sheathing may also be used between the courses of large timbers. The different courses run in different directions; if the roof is of a two-course thickness both courses may run transversely, while if it has three courses the lower and upper courses run transversely and the middle course longitudinally. All caulking of the air chamber is done from the inside of the working chamber, against the air blowing out, while the outside planking is caulked from the outside, to prevent the water from getting in.

The roof of the 102×172-foot caisson for the New York tower of the Brooklyn bridge was composed of a solid mass of squared timbers, 22 feet thick, all timbers being 12×12 inches in section, and thoroughly drift-bolted together. This is the thickest roof that has ever been used.

The roof of the 31×79-foot rectangular caisson for the old piers of the Baltimore and Ohio Railroad bridge at Havre de Grace was composed of eight thicknesses of 12×12-inch timbers, the courses alternating in direction, some running longitudinally, others transversely, and still others diagonally. The lower surface was sheathed with 3×12-inch planks. This form of roof is typical of a number of caissons built under the direction of WILLIAM PATTON, who was an extremist in respect to thick roofs.



The roof of the east abutment caisson of the St. Louis arch bridge was only 4 feet 10 inches thick, the upper three layers being composed of 16×16-inch timbers. The shape of this caisson was an irregular hexagon, with extreme dimensions of 82 by $72\frac{1}{2}$ feet. This comparatively thin roof was made possible by the use of two wooden bulkheads below the roof and two iron girders above, the latter running at right angles to the former, and all supporting the roof. The upper surface of this roof was covered with plate iron,

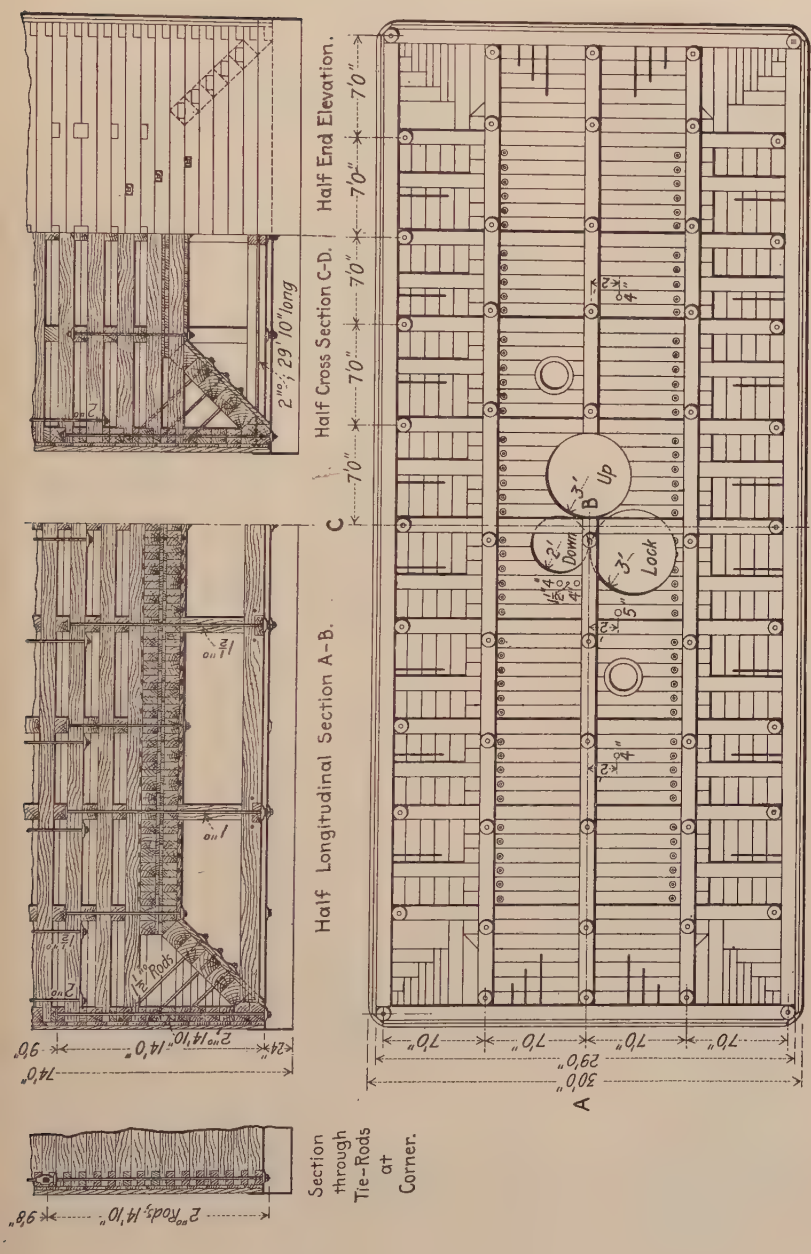


FIG. 93*a*.—Pneumatic Caisson for Pier IV of Bellefontaine Bridge Designed in 1892.

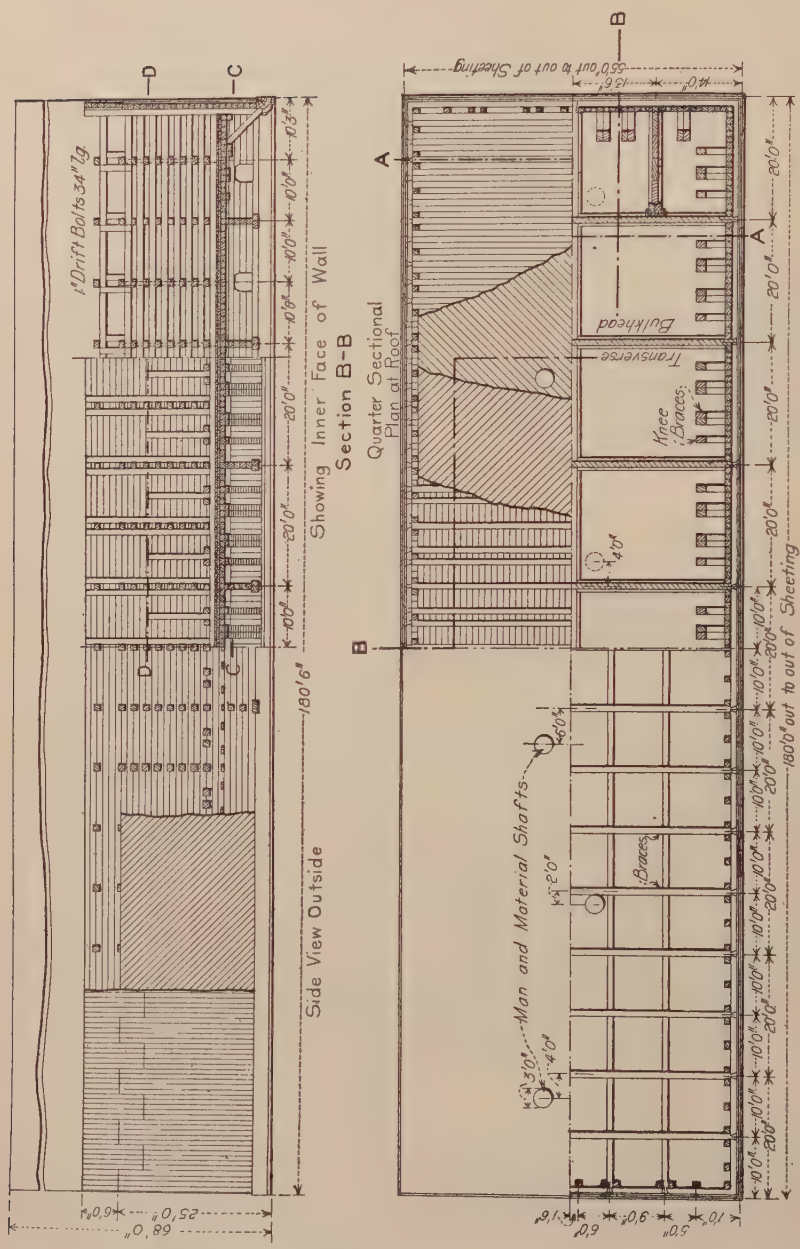


FIG. 93*b*.—Pneumatic Caisson for South Main Pier of New Quebec Cantilever Bridge.

while in the Brooklyn bridge caissons the under side was covered with wrought-iron plates; in both cases this was done to obtain an air-tight roof. It was a very expensive method, since oakum caulking is sufficient. But in the Brooklyn bridge caissons it was done for the added purpose of fire protection, for

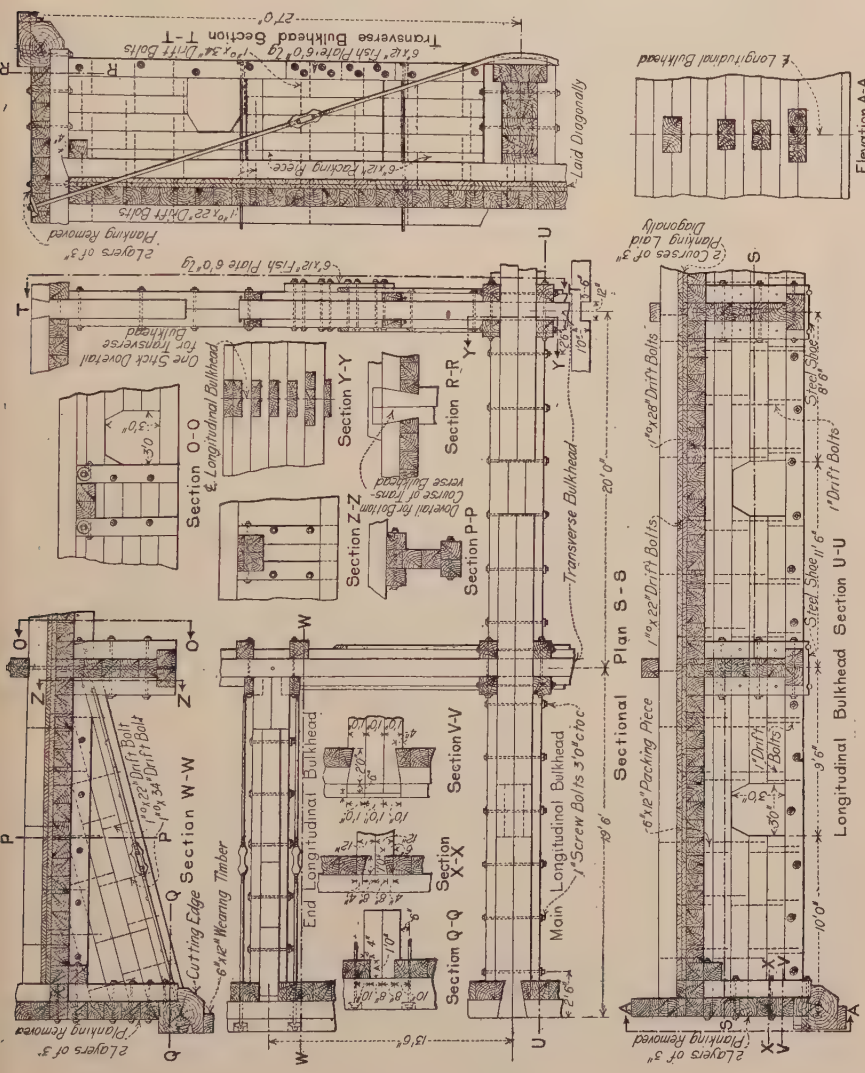


FIG. 93d.—Details of Pneumatic Caisson for New Quebec Bridge.

in those early caissons torches were used for lighting purposes, and as there was always a considerable amount of air escaping between the timbers the danger of fire was very great.

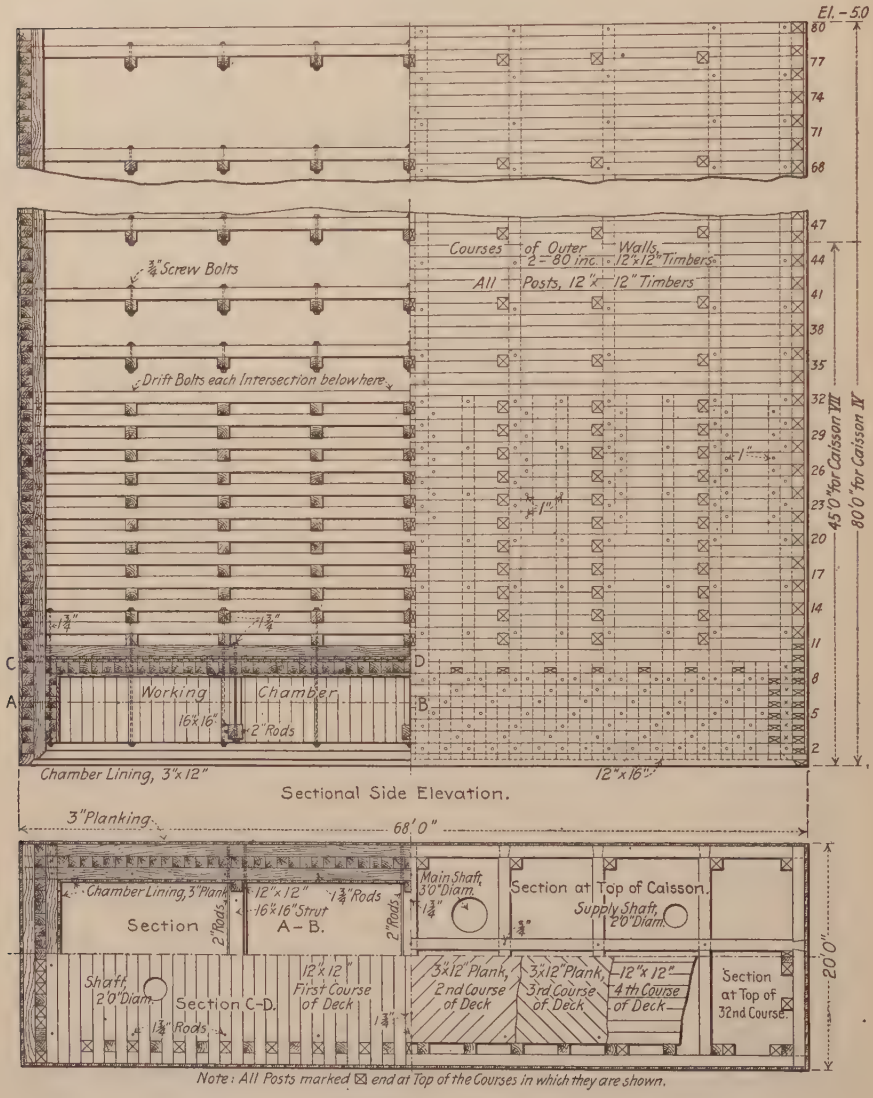


FIG. 93e.—Pneumatic Caisson for Broadway Bridge, Portland, Ore.

In recent years the tendency has been to use more courses of 3-inch sheathing, usually tongue and groove, in order to get a more nearly air-tight roof. As shown in Fig. 93*a* the roof of

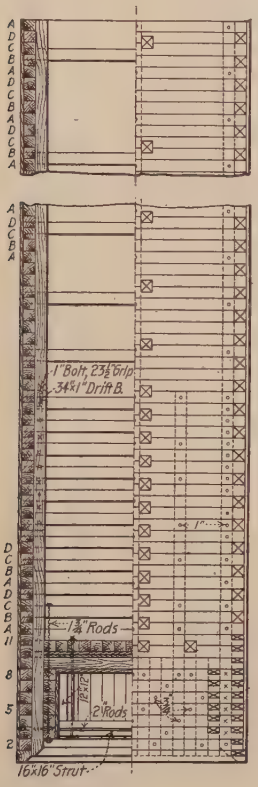


FIG. 93*f*.

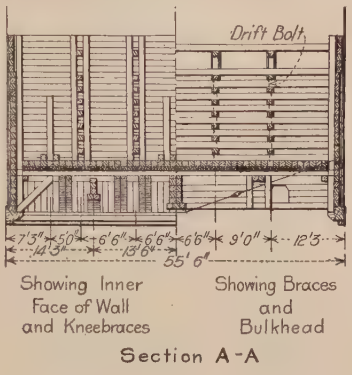


FIG. 93*c*.—Quebec Bridge Caisson.

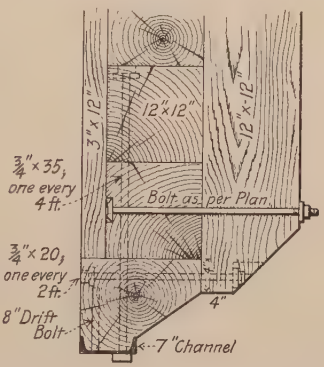


FIG. 93*h*.—Section of Cutting Edge.

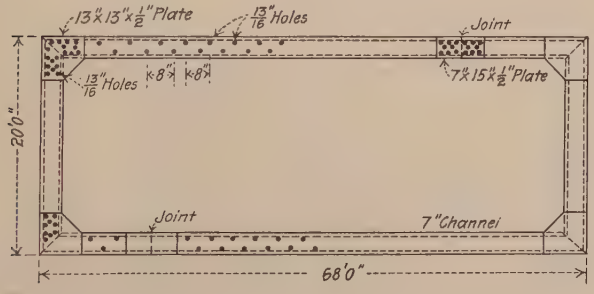


FIG. 93*g*.—Plan of Steel Cutting Edge. Broadway Bridge.

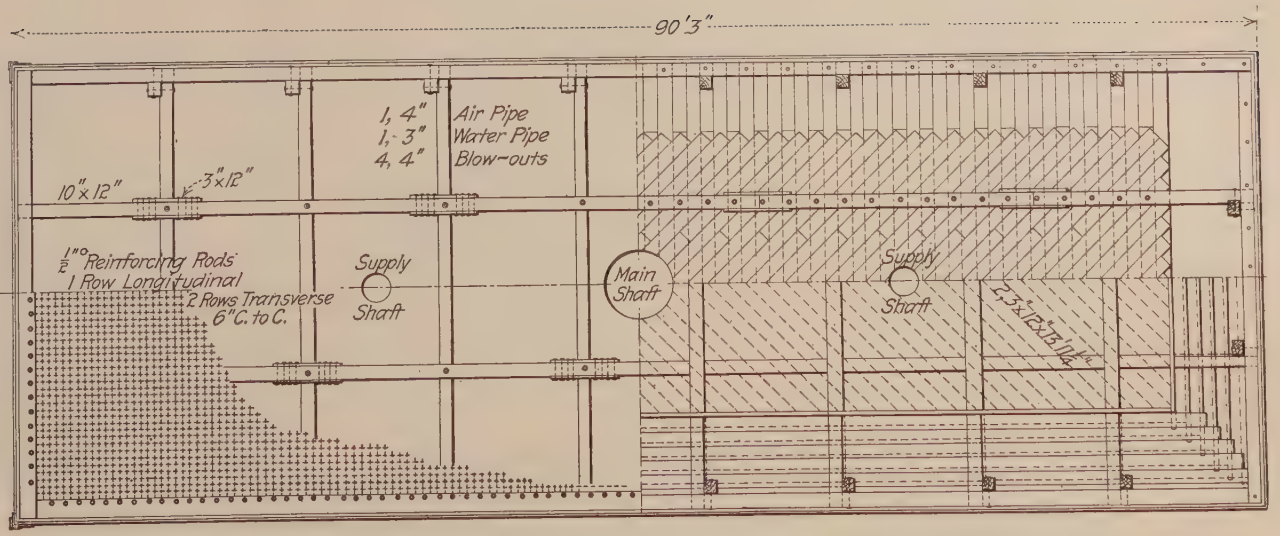


FIG. 93*i*.—Pneumatic Caissons for the River Piers of the Municipal Bridge at St. Louis, Mo., 1910.

The roof of the caissons for the Municipal Bridge consisted of a single solid course of transverse timbers, sheathed on the upper and lower surface with 3×13-inch planks, placed diagonally and well caulked. This roof served as a form for a layer of concrete placed on it and reinforced with two rows of transverse and one row of longitudinal 1-in. round rods, spaced 6 inches center to center, and placed a little above the upper layer of sheathing. Sinking the caisson was not commenced until this concrete had hardened.

the caisson for pier 4 of the Bellefontaine bridge, built in 1892, consisted of two courses of large-size timbers, between which were placed two courses of sheathing, laid diagonally. The lower side of the roof was also lined with sheathing. Another

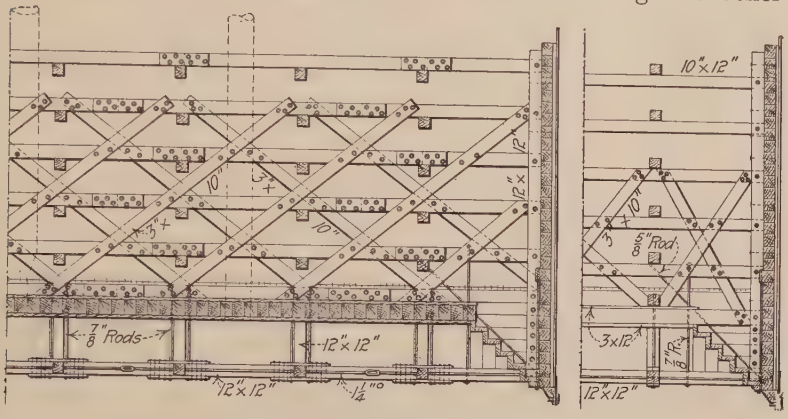


FIG. 93*j*.—Half Longitudinal and Half Transverse Sections.

notable feature of this roof, which is characteristic of many built by Geo. S. MORISON, is the relatively thin roof used. This was made possible by connecting the roof to the bracing timbers of the crib above by means of tie rods.

As shown in Figs. 93*b*, *c*, and *d* the roof of the south main pier caisson of the new Quebec bridge consisted of one solid course of longitudinal and one solid course of transverse timbers, separated by two crossed courses of diagonal 3-inch tongue-and-grooved planks. Here numerous bulkheads made possible a thin roof.

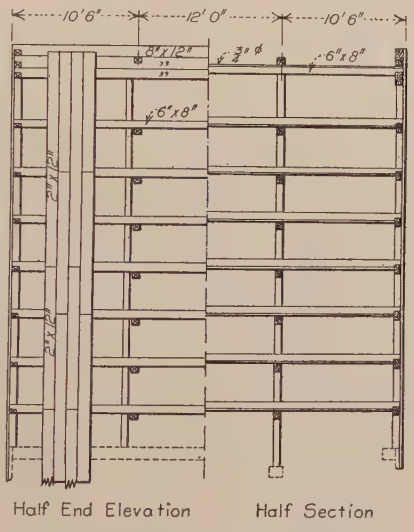


FIG. 93*l*.—Framing of Cofferdam.

The same form of roof was used in caisson 4 of the Broadway bridge, Portland, Oregon, as shown in Figs. 93*e* and *f*.

REINFORCED-CONCRETE ROOFS.—Decreasing the amount of timber in the roof offers two advantages: First it increases the strength of the pier foundation, the timber being the weakest element in the structure; and second, it decreases the compressibility of the foundation. The cost is also greatly lessened.

The description of the roof construction for the Municipal bridge is given under Fig. 93*i*. Another

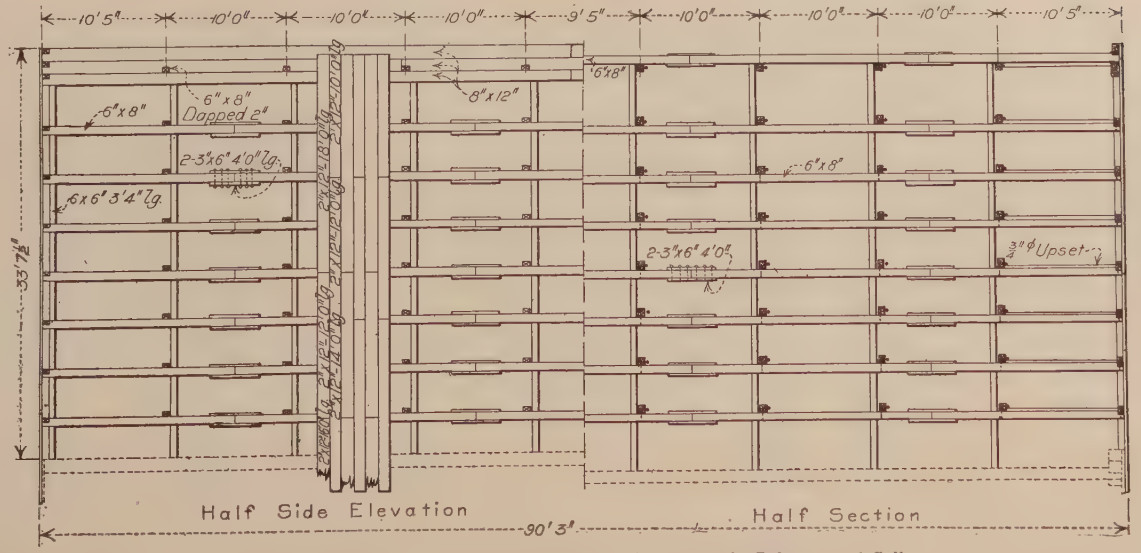


FIG. 93*k*.—Framing of Cofferdam on Top of Pneumatic Caisson and Crib.

example of a reinforced-concrete roof is that for the caissons of the Passyunk Ave. highway bridge piers, across the Schuylkill River, Philadelphia. The largest caisson was 22×60 feet in plan and its roof was reinforced with 1-inch square, twisted horizontal rods running transversely and spaced 12 inches on centers. The thickness of the concrete slab first cast was 18 inches, the forms consisting of a temporary wooden ceiling of 3×12-inch planks.

ART. 94. SIDES OF WORKING CHAMBER

The sides of the caisson should be made strong and rigid enough, not only to take the direct vertical loads, but also to withstand safely sudden lateral thrusts, eccentric loads due to unequal sinking of opposite sides, etc. To prevent leakage of air outward and of water inward all joints should be thoroughly caulked. The necessary thickness of walls will depend somewhat on the clear height of the working chamber, as well as on the kind of material through which the caisson is to be sunk. The clear height should not, however, vary much from 6 feet.

The sides must be vertical. To batter the sides for the purpose of reducing the friction is to invite trouble. Such a design makes it more difficult to sink the caisson plumb, and is apt to increase instead of decrease the friction by allowing boulders to roll into the open space.

Practically all working-chamber sides are constructed of two forms: namely, that in which the vertical section is V-shaped, and composed of two walls; or that in which the vertical section is essentially a rectangle and composed of a single wall. The former has the advantage of being more rigid and so requires less bracing, while the latter has the advantage of permitting excavation under the cutting edge to be more easily made.

In the V-shaped form the space between the outer and inner walls may be built solid with timber, as was done in the east abutment caisson of the St. Louis arch bridge; or it may be made hollow and afterward filled with concrete, as was done

in most of the caissons designed by G. S. MORISON, a typical form of which is shown in Fig. 93*a*. Here the outer wall was made of 12×12-inch timbers, sheathed on the outside with two layers of planking, the outer one running vertically and the inner one diagonally. The inner wall consisted of a single thickness of 17×17-inch timbers sheathed with 4-inch planks and tied to the outer wall with rods.

The St. Louis Municipal bridge caissons, Fig. 93*j*, had outside walls of 10×12-inch timbers, sheathed with two courses of planking: one 3×12-inch, running diagonally, and the other, 2×12-inch, running vertically, the latter being on the outside to reduce friction in sinking. The inner wall was formed of 4×12-inch horizontal planks, stepped and supported at intervals of 10 feet on vertical struts. The small size of material used in this wall was made possible by reinforcing the concrete in the space between the walls. Stepping the wall made it possible to count on the horizontal projection of this inner wall as taking load when the caisson was filled with concrete and in its final position. This cannot be done when the wall is on a slope. A further advantage is that the projections gave better control of sinking, there being less danger of sudden drops than when the wall is sloped.

The rectangular section of side wall is used more widely than the triangular, on account of the facility with which the spoil near the sides may be excavated. Figs. 93*e* and *f* illustrate a good example of this type. It is composed of a double thickness of horizontal 12×12-inch timbers, separated by a single thickness of vertical 12×12-inch timbers, some of which extend up beyond the caisson to form a part of the crib. Both the outside and inside faces of the wall are faced with 3×12-inch planks. Figs. 93*b*, *c* and *d* also illustrate the same type.

ART. 95. DETAILS OF CUTTING EDGE

The cutting edge, as the part of the caisson which rests on the ground is called, must be designed to serve four functions: First, it must be sufficiently strong and tough to stand the

strains and abrasive action of sinking; second, it must be of a form which will allow the caisson to sink readily without excavating under the cutting edge; third, it must have bearing surface enough to prevent sudden sinking when a soft stratum is encountered; and fourth, it should be so designed that air cannot readily escape under the same. To fulfill the first requirement the cutting edge is usually made of some tough and strong wood, such as elm, or else is shod with a metal plate or piece of tough wood. The second and third are conflicting requirements; for the second a true knife edge is the ideal form, while for the third a considerable breadth of bearing is desirable. As constructed, the width will vary from about 4 inches to 18 inches. To meet the fourth requirement, a vertical plate extending about 6 inches below the cutting edge is often used. Where the soil is dense this plate may be dispensed with.

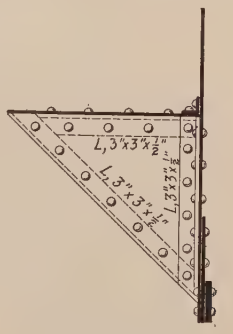


FIG. 95*a*.—Details of Cutting Edge.

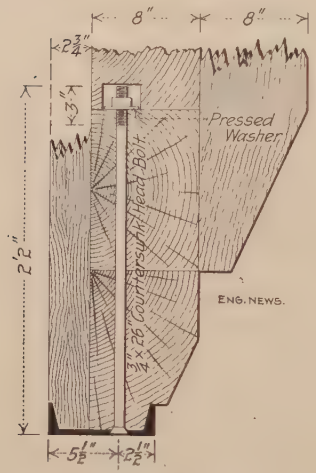
Many engineers at present favor the blunt cutting edge in preference to the sharp one. T. K. THOMSON'S experience is, that where the knife edge is needed, *i.e.*, in hard material, to allow getting close to the outside edge for excavating, it would cost too much to make the cutting edge strong enough, and where the material is soft a knife edge is not needed.

Fig. 93*d* illustrates the use of a timber wearing plank on the cutting edge. It was 6×12 inches in section, the main timber forming the cutting edge being 30×30 inches in section, while the upper inner corner of the latter was rebated 9 inches to form a seat for the feet of the vertical wall timbers. The advantage of a timber over a metal cutting edge lies in less time being required to obtain it, and in the greater ease with which it may be replaced when broken.

The form of cutting edge used by G. S. MORISON, illustrated in Fig. 95*a*, consisted of a horizontal and vertical plate, the latter being stiffened at intervals and fastened to the horizontal

plate by steel diaphragms, which are stiffened on the edges by three angles. The horizontal plate extended under, and was fastened to both the lower surface of the bottom timbers and the lower edge of the outside sheathing, while the vertical plate was fastened to the same outside sheathing. Near the bottom the vertical plate was reinforced with two others. Fig. 93*a* shows the appearance of this cutting edge in place.

The bottom timber of the caisson shown in Fig. 93*f* extended out beyond the timbers above to protect the lower edges of the outside sheathing, while it in turn was protected by steel plates on all sides but the top. This form of construction, having a vertical plate on the outside and a horizontal angle with its vertical leg down and fastened by rivets to the vertical plate, and with its horizontal leg fastened to the lower surface of the lower timber, is widely used, but is not economical.



Detail of Cutting Edge. (Enlarged)

FIG. 95*b*.

The cutting edge of the caisson used in the Kinzie St. draw-bridge, Chicago, was formed with an 8-inch channel iron laid horizontally with flanges turned up as shown in Fig. 95*b*. The same general form was used on the Broadway bridge caissons (Fig. 93*h*), the only difference being that in the latter case the cutting-edge timber extended out to protect the bottom of the sheathing, while in the former case the channel iron served this purpose. This form of metal cutting edge is the most economical and was designed in 1901 by T. K. THOMSON.

ART. 96. BRACING OF CAISSON

Every caisson requires more or less bracing; the larger and higher it is the more bracing will it require. This bracing may

be in the form of struts and tiers near the bottom, running horizontally the length and breadth of the caisson, or it may be in the form of bulkheads, or trusses. The latter two usually serve the added purpose of supporting the roof.

The bracing in the 33×90-foot caisson of the St. Louis Municipal bridge, shown in Figs. 93*i* and *j*, consisted of eight transverse and two longitudinal lines of horizontal 12×12-inch struts spaced about 10 feet apart, with $1\frac{1}{4}$-inch adjustable rods on both sides of each strut. The struts at their intersections were braced with vertical 12×12-inch timbers and pairs of $\frac{7}{8}$-inch rods extending to the deck of the caisson. A similar form of bracing was employed in the Bellefontaine bridge caissons, as illustrated in Fig. 93*a*, as well as in the Broadway bridge caissons, Figs. 93*e* and *f*.

The south main pier caisson of the New Quebec bridge, 55×180 feet in plan, was divided by timber bulkheads, as shown in Figs. 93*b* and *c*, into eighteen rectangular compartments approximately 19×25 feet in size. These longitudinal and transverse bulkheads were respectively 24 and 12 inches thick, except the lower course which was 12 inches thicker. All extended from the ceiling to about the top of the cutting edge. Each transverse bulkhead was trussed by a pair of adjustable diagonal rods, the ends of which took bearing in the end walls at roof level, through beveled washers; in the center they bore on steel plates, the latter in turn bearing on both longitudinal and transverse bulkheads. The end walls on each side of the longitudinal bulkhead, were braced by a solid-web knee brace 12 inches thick, reaching from the cutting edge to the top of the first transverse bulkhead. Between bulkheads the sides were knee-braced to the roof by single and double 12×12-inch struts inclined at an angle of 45 degrees.

The bulkheads of the east abutment caisson of the St. Louis arch bridge were of very massive construction, being made of eight horizontal courses of timber, the upper course having eight timbers in it, making a width of 10 feet, while the bottom course had three timbers, making a width of $3\frac{1}{2}$ feet. The numbers varied in the horizontal courses between these two

values in such a way as to give a V-shaped section of bulkhead. The height was 9 feet.

A longitudinal wooden truss was used to brace the 31×79-foot caisson of the Havre de Grace bridge. It was 6 feet deep, the upper and lower chords being composed of two pieces of 12×12-inch timbers. The web members, both vertical and diagonal, were composed of timber struts and diagonal rods, the latter extending through the first deck course of the caisson. Cross braces were placed between the bottom chord of the truss and the side walls.

ART. 97. CRIB CONSTRUCTION

Some writers consider the crib as a part of the caisson, but since the crib may sometimes be dispensed with and the pier built directly on the caisson, it will avoid confusion by separating the two. A certain height of crib is often built as an integral part of the caisson to facilitate floating the structure into place. The purpose of the crib is two-fold: First, it serves as a form for the concrete; and second, it serves temporarily as a cofferdam to keep out the water. If the masonry or concrete work is kept sufficiently in advance of the sinking the crib may sometimes be dispensed with, but this is sledom done because it brings too much weight on the caisson. The crib is a permanent part of the foundation and usually its walls are a continuation of the walls of the caisson, perhaps slightly modified. The crib is thoroughly braced with longitudinal and transverse timbers left permanently in place.

Although it is customary to fill the crib with concrete, yet under some circumstances this may not be done. In the substructure for Pier 2 of the Memphis bridge, where the nature of the soil made it necessary that the load on the foundation bed be kept down to a minimum, the pockets near the walls in the crib were left empty, while for about 15 feet down from the top of the crib a solid timber grillage was used, thus decreasing the weight of the structure very considerably.

The crib for the south main pier of the New Quebec bridge

had a wall made of a single thickness of horizontal 12×12-inch timbers to a distance of 25 feet above the cutting edge of the caisson, braced by inside vertical 12×12-inch timbers, spaced as shown in Figs. 93*b* and *c*, the latter being extensions of certain of the vertical timbers forming the sides of the caisson. The outside was sheathed with the same material as used for the caisson. The walls were braced with horizontal longitudinal and transverse struts 24 inches apart vertically, up to a height of 25 feet above the cutting edge of the caisson, dividing the crib into ninety pockets approximately 10 feet square. A similar bracing course was placed 29 feet above the cutting edge of the caisson; above this point there was no bracing, it being replaced with a concrete retaining wall reaching to the top of the crib, built against the walls of the latter and battered on the interior face, increasing in thickness from the top down. This was placed early in order to allow it to harden before any stress was put upon it. The advantage of this retaining wall is that it made the upper part of the crib a solid monolithic mass of concrete.

The crib shown in Fig. 93*a* had the bracing carried to the top and was notable on account of the manner in which the bracing was tied together with vertical rods. Here the lower courses of bracing helped to carry the roof loads; for this reason the part of the crib up to the top of the rods passing through the roof may be considered a part of the caisson.

The walls of the cribs for the St. Louis Municipal bridge piers consisted for the most part of one thickness of 10×12-inch timbers, sheathed on the outside with one layer of 3-inch diagonal and one layer of 3-inch vertical planks. The bracing consisted of vertical 12×12-inch timbers and of eight rows of horizontal transverse and two of horizontal longitudinal 10×12-inch timbers. As shown in Fig. 93*j* a large amount of 3×10-inch diagonal bracing was also used, giving a truss-like action to the bracing and greatly strengthening it.

The crib construction of the Broadway bridge is shown in Figs. 93*e* and *f*; the detail are so simple that no explanation is necessary.

ART. 98. COFFERDAM CONSTRUCTION

Both durability and appearance require that no part of the crib extend above low-water level; and moreover, to keep the obstruction to the current as small as possible, the crib is stopped and the pier commenced at a considerable distance below low water. In some cases, where the current has a high velocity, the pier is started at or below the river bed, or the upper part of the crib is built with pointed ends. For these reasons, unless conditions are such that the pier construction can be kept well above water-level, a cofferdam in which to build the pier becomes necessary. Ordinarily cofferdams may be dispensed with only when the construction is carried on at low water stages or when the friction and resistance to sinking is large. As a general rule it is desirable to keep the weight on the caisson as small as possible as this affords better control of the sinking. Even when possible many engineers prefer not to start building the pier until the caisson is sunk to final position, for only at such a time can the masonry be started in the correct position. The walls of the cofferdam are usually made of lighter construction than those of the crib, but it is always thoroughly caulked, and braced by struts running the length and breadth of the structure. As the pier is built up these braces are removed and the walls are braced against the pier. On the completion of the latter the cofferdam is removed, if not the whole structure, at least that part above low water.

Figs. 93*k* and *l* illustrate the cofferdam used for one of the piers of the St. Louis Municipal bridge. The left dotted lines represent the top course of crib and the right dotted lines the top of struts. The cofferdam, which was 33 feet $7\frac{1}{2}$ inches long, consisted of a frame of horizontal 6×8-inch and vertical 6×6-inch timbers, sheathed with 2×12-inch planks. It was braced with 6×8-inch struts, 4 feet apart vertically, and in rows about 10 feet apart horizontally.

The cofferdam used for the Brooklyn pier of the Manhattan bridge, New York, N. Y., was one of the highest that has ever been used in pneumatic caisson work, being 44 feet high and

about 75×144 feet in plan. It was built in three sections, the sides of the first two sections being made of 10×12-inch horizontal timbers laid close and supported by 12×18-inch verticals, spaced 12 feet apart. On the outside two layers of 3×12-inch sheathing were placed, the inner planking being horizontal and the outer vertical. The upper section differed from the others only in having 8×12-inch instead of 10×12-inch horizontals.

ART. 99. PNEUMATIC CAISSONS OF CONCRETE

Pneumatic caissons built entirely of concrete have been used to some extent in Europe, but in this country the nearest approach to the all-concrete pneumatic caisson are those for the Beaver bridge, described in Art. 90. As there explained most of the sinking was done by the open-well method. With the exception of a very few cases, like the one just noted, the tendency in this country has been to use wood, but at the same time to decrease the amount formerly used by reinforcing the lower part of the crib concrete, as was done in the St. Louis Municipal bridge caissons. A covering of timber offers three advantages: First, it avoids the necessity of waiting for the concrete to harden before commencing sinking operations; second, it offers less resistance to sinking because of the reduced friction on the sides; and third, it forms a protection in sinking for the concrete of the sides.

The pneumatic process was used during the final part of the sinking of the Beaver bridge caissons in order that the bottom might be thoroughly cleaned, as well as to permit laying the concrete filling in air. The caisson was changed from the open to the pneumatic type in the following manner: It was first freed of water down to a level which permitted the placing of horizontal wooden frames in each of the wells at an elevation of about 9 feet above the cutting edge. Concrete was then placed on these forms, filling the wells, the first 7 feet being allowed to harden for a week before placing the rest. At the center of each well a vertical shaft, 3 feet in diameter, was

placed to form a means of communication between the working chamber and the outside.

ART. 100. PNEUMATIC CAISSONS OF METAL

The abundance of timber in America has limited the use of the metal type to relatively few cases, while in Europe it has been extensively used.

The river piers of the St. Louis arch bridge, the first structure in this country founded on large pneumatic caissons, rest on metal caissons. Two reason may be given for this fact: First, there was considerable uncertainty as to the action of a timber roof when subjected to the horizontal thrust from the superstructure; and second, timber had not been used in caisson construction to serve as a precedent.

The caisson for the east pier, which was hexagonal in plan, with over-all dimensions of 60×82 feet, had walls of wrought-iron plates $\frac{3}{4}$ inch thick, braced with iron brackets extending from the bottom to the top, and spaced $2\frac{1}{2}$ feet apart. The roof was formed of $\frac{1}{2}$-inch iron plates riveted to the lower flanges of thirteen parallel iron girders, spaced 5 feet 6 inches apart. It was also supported by two heavy bulkheads of oak timber, 7 feet high, in the air chamber. These strong supports for the roof were necessary because the latter had to take the entire weight of a 100-foot height of stone masonry.

The walls of the caisson extended above the roof to form an enclosure, in which the masonry was laid. No monolithic concrete was used in this structure. For some distance up the masonry covered the entire cross-section of the crib, but above this it was stepped off, the space between the iron envelope and the masonry being braced with timbers and filled with sand. For the west pier caisson the iron envelope was carried up but 20 feet, after which the masonry was laid in the open, care being taken to keep the top of the same above water-level.

The metal pneumatic caissons for the Alexander III bridge, Paris, France, built in 1897, are among the largest of any type ever used. In plan one caisson had the shape of a parallelogram

(the angle being 84 degrees), the length of the sides being approximately 145 and 110 feet, transversely and parallel, respectively, to the axis of the bridge. The working chamber had a clear height of 6.23 feet and through this extended four transverse girders, each 6.23 feet high, their bottoms forming cutting edges, and dividing the chamber into five subchambers. On their upper flanges these girders supported twenty-seven longitudinal girders, 5.2 feet deep, which carried the roof of the steel-plate platform that formed the deck of the caisson proper. The transverse girders had solid-plate webs for nearly one-third of their length at each end and open web members in the central part. The longitudinals were ordinary latticed girders. The working chamber had a roof of steel plates 0.2 inch thick which were fastened to the lower flanges of the longitudinal, and to the upper flanges of the transverse girders. These plates did not extend horizontally through to the vertical sides of the caisson, but at the sides followed down the inclined end posts of the transverse girders, and at the ends followed the knee braces down to the cutting edge to give sloping inside walls on all four sides.

Between these inclined plates and the outer vertical walls was a triangular space filled with concrete. The outside wall plates and the transverse girders were all stiffened with knee braces extending from the cutting edge to the longitudinal girders.

The outside wall plates were reinforced on the lower edges by an outside vertical plate and the vertical flange of an inner angle, while the transverse girders were reinforced for bearing and cutting strains by adding two angles riveted, with their horizontal flanges upward, to the lower edge of the vertical web plate of the lower chord. The cofferdam above was 19.7 feet high and was composed of riveted and caulked vertical plates, 0.118 inch thick, with a light angle-iron frame and light inclined angle-iron struts from near the upper edge and the middle of the top of the transverse girders. The total distance sunk was 27 feet below ordinary water-level. For further details the reader is referred to either Engineering News, vol 39, page 254,

April 21, 1898, or Engineering Record, vol. 37, page 275, Feb. 26, 1898.

ART. 101. CYLINDER PIER CAISSONS

The foundation for a cylinder pier is often placed by the pneumatic process, in which case, like the open-cylinder caisson, there is usually no particular point at which the caisson may be said to end and the pier begin. The pneumatic cylinder caisson is very similar to the open caisson in many cases, the only difference being that the former is fitted with horizontal diaphragm doors to form the air-lock. Often a part of the sinking is done by the open-caisson method and the remainder by the pneumatic method. As noted in Art. 92 the cylinder caisson was the first type of foundation to which the pneumatic process of sinking was applied in this country.

Fig. 101*a* illustrates the cylinder piers and pneumatic cylinder caissons used for the Columbia River bridge at Trail, B. C. The shells were of steel plates from $\frac{5}{16}$ to $\frac{7}{16}$ inch thick. The lower 61 feet were formed of a double shell, the diameter of the inner shell being 3 feet, and that of the outer one 9 feet at the bottom and 6 feet at the top. Beginning at a point 8 feet above the bottom of the caisson the inner shell was splayed out to meet the outer shell at the cutting edge, thus forming a working chamber 8 feet high. Near the bottom the two shells were braced together with diagonal lacing as shown in the diagram.

The upper parts of the cylinders were connected and braced by two vertical transverse $\frac{5}{16} \times 60$-inch plates, 2 feet apart, braced together and the space between the two filled with concrete.

The air-lock was formed by placing two diaphragm doors in the inner shaft, one about 13 feet above the cutting edge and the other at a point about 16 feet higher. As sinking proceeded, a third door, about 16 feet above the second door, was added, the upper two doors being used to form the lock, while the lower door was used for emergencies. These caissons were designed

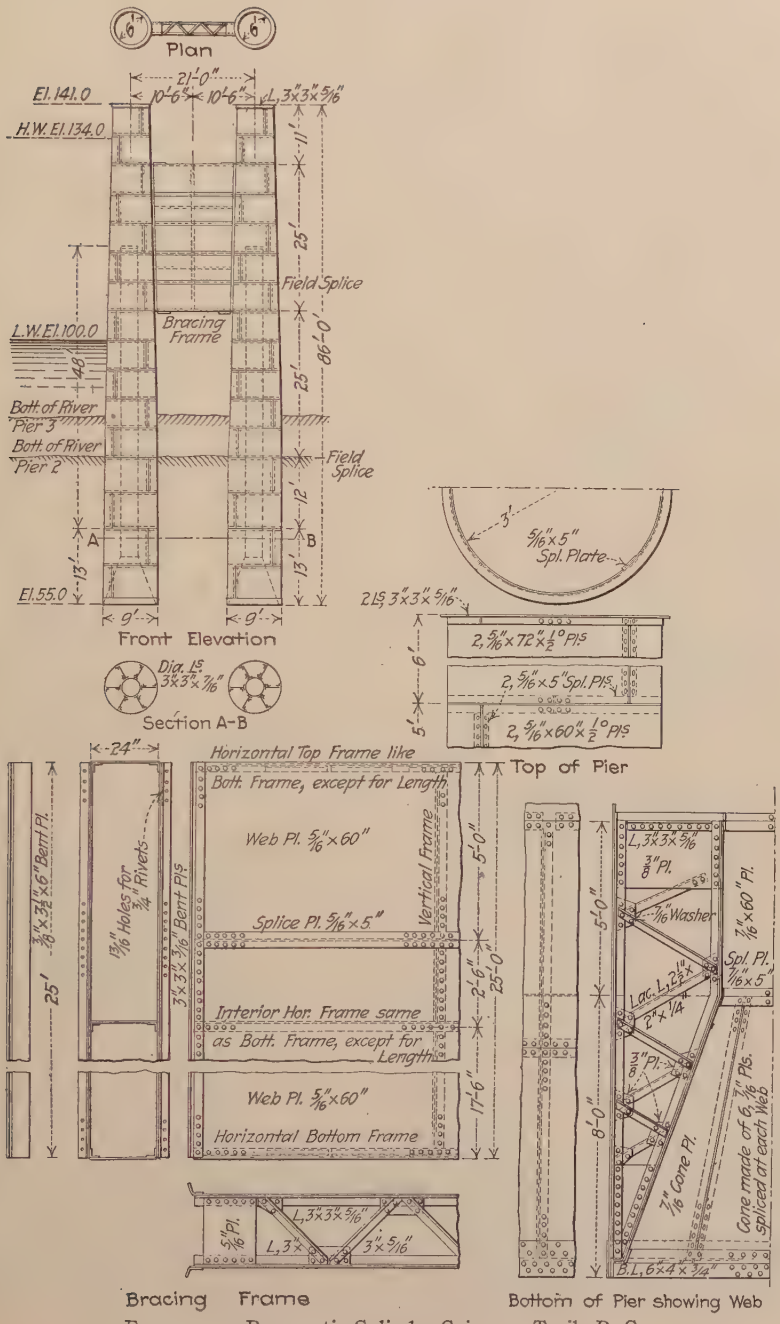


FIG. 101a.—Pneumatic Cylinder Caissons, Trail, B. C.

by WADDELL & HARRINGTON, and may be considered to represent current standard practice.

In the repairs of the Atchafalaya River bridge, each pier consisted of a pair of 8-foot diameter steel cylinders, filled with concrete and braced together at the top by a stiffened web plate or diaphragm about 20 feet high, as shown in Fig. 101*b*. Each cylinder had, in addition to the outer 8-foot diameter

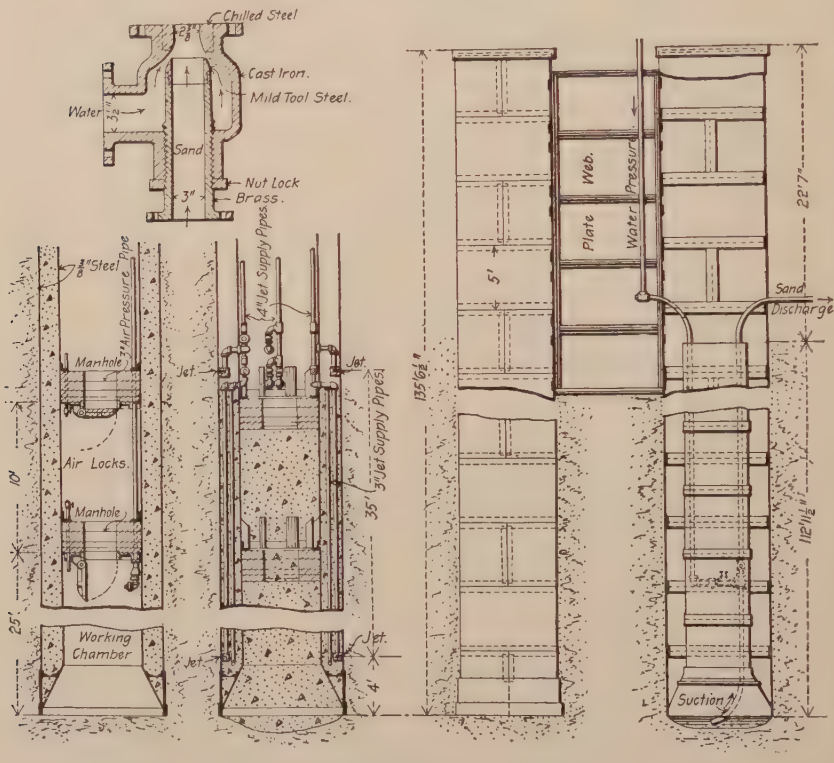


FIG. 101*b*.—Pneumatic Cylinder Caissons, Atchafalaya River Bridge.

shell, an inner concentric shell 5 feet in diameter, with a conical section uniting it with the cutting edge and closing the lower end of the annular space between the two shells. The shells were connected by four stiff webs. The inside shell terminated about $22\frac{1}{2}$ feet below the top of the outer one, the latter having a total length of over $135\frac{1}{2}$ feet and was made with

5-foot rings erected in 10-foot sections. The working chamber was 25 feet high, and had a roof consisting of a 2-foot oak diaphragm made of four thicknesses of timber, with a circular hole 2 feet in diameter closed by a cast-iron door.

In the piers of the Glasgow bridge, which were sunk by the pneumatic process the diameter of the outer shell was 15 feet, the thickness of the shell at the base being $\frac{1}{2}$ inch and at the top $\frac{5}{16}$ inch. The shaft which was 3 feet 7 inches in diameter formed the inner cylinder, and this was removed before filling the working chamber and air-shaft.

Almost no records exist of the use of the reinforced-concrete pneumatic cylinder caisson. In Art. 102 there is given an example of this type, in which the first part of the sinking was done by the open-caisson method and the latter part by the pneumatic process.

Art. 102. Combination Cylinder Caissons

With the cylinder caisson it is a simple matter to construct the cylinder to be used either as an open or a pneumatic caisson. This makes it possible to utilize the advantages of both methods of sinking, the open caisson being used for that part of the sinking in which the material can be dredged or pumped out, and the pneumatic process for that part where boulders or compact material is met with, and in finally preparing the foundation bed and placing the concrete filling in the working chamber.

The caissons for the Merrimac River bridge, between Salisbury and Newburyport, Mass., were of this type. Each caisson consisted of an 8-foot diameter cast-iron shell, the metal being $1\frac{1}{2}$ inches thick and cast in 8-foot sections. These sections had inside flanges bolted together and a mixture of red lead and linseed oil was placed between the joints.

The cylinders were sunk by inside dredging to a layer of boulders and gravel. They were then loaded with pig iron, air-locks placed on top, and air pressure applied. No attempt was made to sink the caissons through the boulders, but instead a novel method was used to transform this boulder and gravel

layer into a good foundation bed. The pressure in the cylinder was reduced a little allowing about a foot or more of water to rise. Portland cement was then mixed with the water to form a grout, which was kept well stirred while the air pressure was increased to force the grout into the gravel. On completion of the grouting a depth of from 10 to 20 feet of 1–2–4 concrete was laid under air pressure, and allowed to harden, after which the remainder was laid in the open.

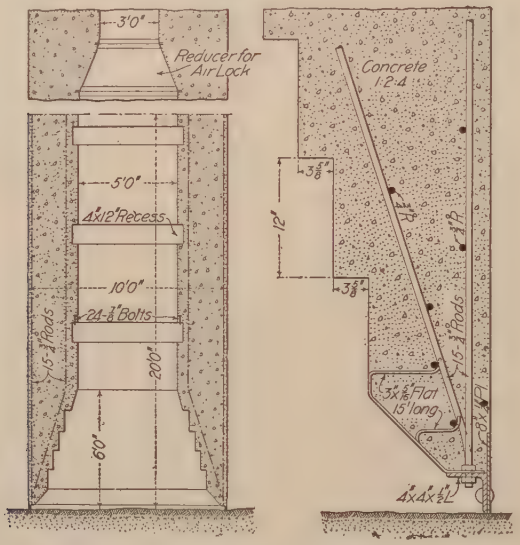


FIG. 102*a*.—Pneumatic Caissons of Reinforced Concrete for Bronx Viaduct of New York Connecting Railway

Fig. 102*a* shows the main details of concrete cylinder caissons used for foundations of the Bronx viaduct of the New York Connecting Railway. The caissons varied from 10 to 18 feet in diameter and were sunk to a maximum depth of 55 feet. The cutting edge was formed of a steel angle and steel plate, and the concrete composing the caisson was well reinforced with vertical and horizontal rods. When sinking through clay the open dredging process was used, while in passing through quicksand air-locks were placed in the upper part of the shafts and the pneumatic process used.

CHAPTER IX

PNEUMATIC CAISSONS FOR BRIDGES

Art. 103. Shafts and Air-Locks

The shafts, which form the means of communication between the working chamber and the outside, are circular in shape and in most cases are of steel plate $\frac{3}{8}$-inch thick; and in sections about 10 feet long, each section being flanged and bolted to the one above and below. Separate shafts are ordinarily used for men and materials, those for the men being about 3 feet in diameter, although if an elevator is used they are often as large as 6 feet in diameter. The shafts for the removal of spoil are about 2 feet in diameter. Where the depths are only moderate it is customary to have a ladder built in the shaft used by the men, but when the depth is considerable a power elevator should always be employed as it is extremely exhausting to climb a long distance after working under high pressure. The men often use the excavating bucket as an elevator.

As explained in Art. 92 the air-lock is a chamber having two doors, one of which opens to the atmosphere and the other to the working chamber. These doors are so placed that the unequal air pressure will always force them against their seats, which have rubber gaskets to prevent the escape of air. The operation of the lock for men is as follows: The lower door being closed and the upper one open, a man enters; the upper door is then closed and compressed air slowly admitted to the lock, and as soon as the pressure in it becomes equal to that below, the lower door opens allowing the man to enter the working chamber.

The air-lock may be of any shape and of any desired size, the latter depending on the number of men or the amount of material it is desired to lock through at a time. The material lock is often but a section of the shaft.

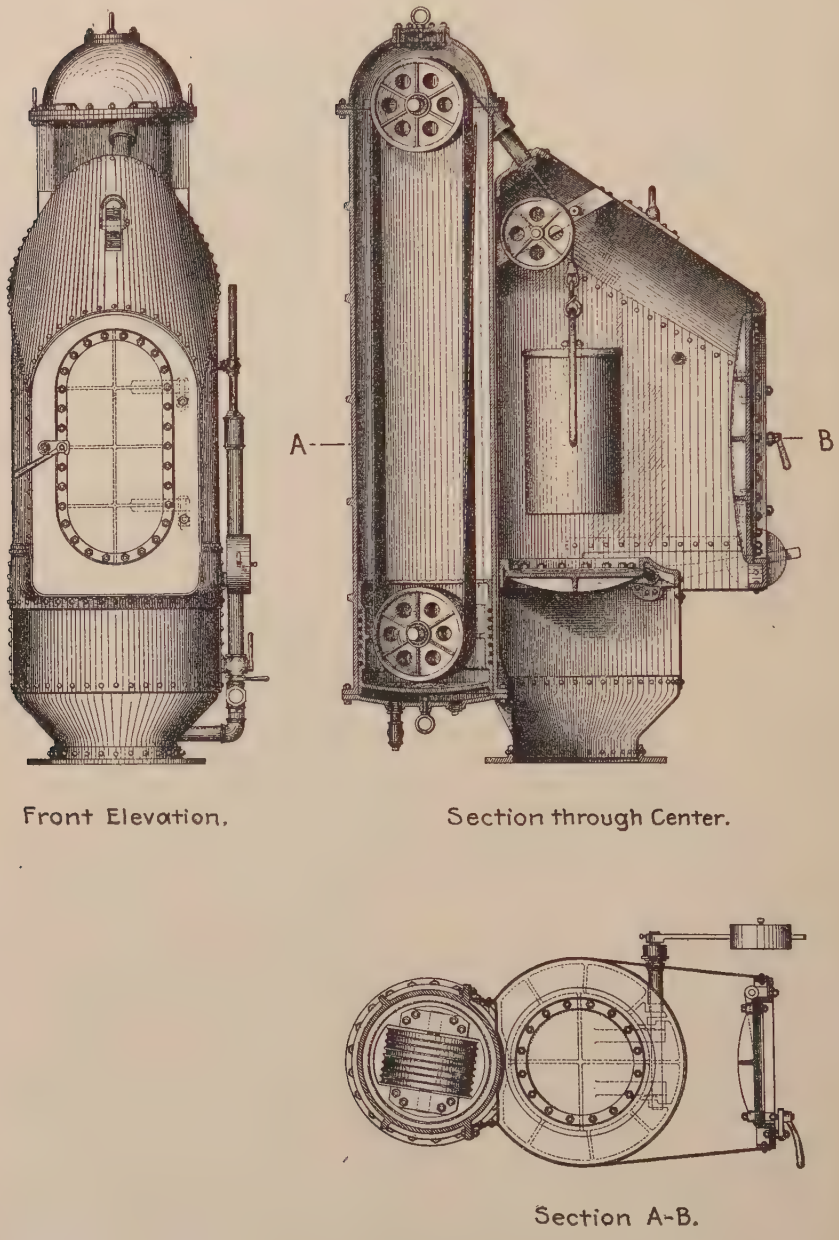


FIG. 103*a*.—Material Lock used in Pneumatic Caissons of Memphis Bridge, 1891

In the early caissons the lock was placed at the bottom of the shaft and extended down into the working chamber, but at present the material lock is always placed at the top of the shaft, while the man lock is placed either at the top or some distance up from the bottom. Caisson sinking with the lock at the bottom is a risky undertaking because a 'blow-out,' that is, a sudden outrush of air, will cause a like inrush of water accompanied by a rapid sinking of the caisson, which is almost sure to damage the lock. With the lock out of commission the men in the working chamber have no chance to escape, while if the lock is at the top the men can climb up and take refuge in the shaft above the level of the water. About the only disadvantage in having the lock on top of the shaft lies in the necessity of removing it each time a new section is added to the shaft; but with properly designed connections this can easily be done, and without dan-

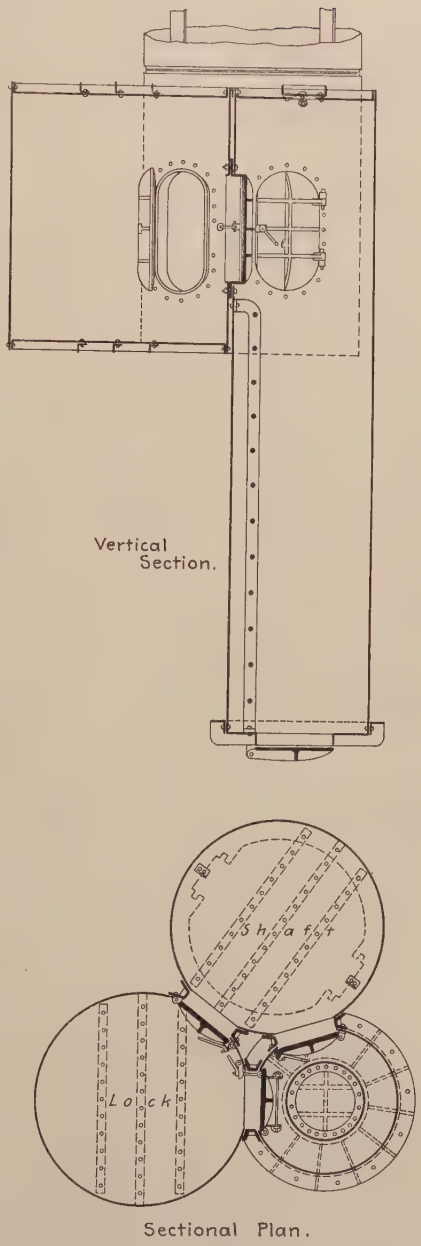


FIG. 103*b*.—Air Lock for Men, Memphis Bridge.

ger, by having an auxiliary door fitted to the lower end of the shaft in the roof of the working chamber which is closed when the lock is taken off.

Two forms of air-locks extensively employed for caissons used for the foundations of buildings are illustrated and

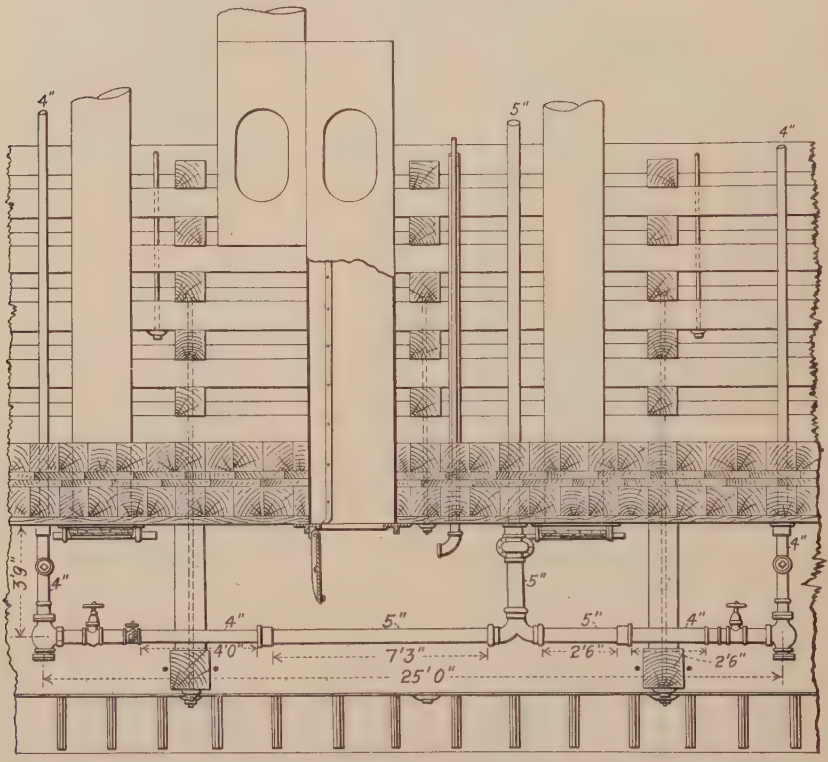


FIG. 103*c*.—Arrangement of Air Lock, Shafts, Pipes, etc. Bellefontaine Bridge.

described in Art. 119. The particular advantage which these types possess is that the bucket may be lowered into the air chamber, filled and taken out without detaching from the hoisting rope.

Another form of material lock which has been employed is illustrated in Fig. 103*a*, this particular one being used on the Memphis bridge caissons. The method of operation is described in Art. 107. The essential difference between this

and the types described in Art. 119 lies in the fact that here the upper door, instead of being in a horizontal plane, lies in a vertical plane at *B*. This necessitates either dumping the material out on being brought to the top or else the bucket must be detached from the cable and taken out.

The form of lock for men employed on the above mentioned bridge is illustated in Fig. 103*b*. It is shown in position in Fig. 103*c*. [1]"The upper shaft through which the elevator-cage runs is a cylinder 6 feet in diameter, the air-lock itself is a cylinder 6 feet in diameter, and the shaft leading to the caisson, a cylinder 4 feet in diameter; the three cylinders are tangent to each other, and the shells are connected by cast-iron door frames carrying doors, while a fourth door opening outward was placed at the bottom of the lower shaft; in working, the door between the two shafts was always kept closed, and the door at the bottom of the bottom shaft was always left open; it was possible, however, if an emergency had arisen to use the lower section of the shaft as an air-lock in itself; when the filling of the working chamber was completed the bottom door was permanently closed."

ART. 104. DESIGN OF CAISSONS

It is impossible to compute even approximately the stresses in the various parts of a caisson and for this reason it is best largely to follow precedent. Engineers who are experts on caisson work, have built many caissons and by observing the weak points have developed strong structures with increasing economy. The examples given in the preceding articles are representative of the best forms in use, and are recommended to the careful consideration of engineers interested in this subject. For more extended information the reader is referred to the bibliography in Chap. XIX.

T. K. THOMSON, a consulting engineer who has specialized in pneumatic caissons, writes on their design as follows:

[1] The Memphis Bridge, by GEO. S. MORISON.

[1] "It is necessary to use considerable common sense and experience in attempting to calculate the strains in a caisson. As regards the deck, for example, it is very easy to calculate the weight to be carried by the deck and the strains that would result therefrom, and we know that the air pressure acting up against the roof will counterbalance a great deal of this weight, making it, in fact, something like a pontoon floating in the water. But on the other hand, the air pressure is often slacked down to almost nothing in order to overcome the friction, and is raised again before much water has time to enter the working chamber; and sometimes an accident to the air plant will suddenly cut off the supply of air, throwing a tremendous strain on the roof. If the principal weight on the roof is concrete it will in many cases be self-sustaining unless too fresh.

"The same with the sides. If the material were absolutely homogeneous all around and the caisson were sunk absolutely plumb, which almost never happens, and the air pressure were kept just equal to the outside pressure, then we would have practically no strain on the sides—but all practical caisson men have seen the sides of caissons collapse, and some very strongly built ones at that. A very much more frequent cause of accident than loss of air pressure is to strike some obstruction on one side, deflecting the cutting edge, and thus throwing much of the weight of the caisson on the weakened side, making bad worse. . . .

"In building wooden caissons I very seldom halve the timbers or use dovetailed joints, preferring to use butt joints as much as possible with plenty of drift bolts. The trouble with butt joints, however, is that while a carpenter will make a dovetail or half-lap joint fit he will probably leave an inch or so play in a butt joint.

"The deck timbers, as well as those in the sides, should be planed on one side and one edge, for the sizes would otherwise vary too much to get a good job, while the planking for the outside and inside of the air chamber should be either tongue and groove, or the sides should be planed for a caulking joint.

[1] See "Construction," Nov., 1908.

The plank should, of course, have its faces also planed." Since very many drift bolts are required in fastening together the heavy timbers in wooden caisson construction, it is desirable to adopt the proper diameter of holes to be bored. For the results of experiments on the holding power of drift bolts and the best ratio of the diameter of hole to that of bolt, see Art. 10 in JACOBY'S Structural Details.

ART. 105. BUILDING AND PLACING THE CAISSON

The caisson may be built on ways on the shore; on pontoons anchored near the shore, or over the site where it is to be sunk; or on a temporary platform supported by piles. Of the three methods, building on ways on the shore is the most widely used, but to make this method satisfactory the following conditions must obtain: First, there must be deep water near the shore; second, the soil must be sufficiently firm to hold the caisson, either with or without the use of bearing piles; third, there must be no danger of a high and rapid rise in the river; and fourth, the shore must not be at a great distance from the site of sinking.

Where satisfactory shore conditions do not obtain and where the water is deep and subject to sudden rises the pontoon method is the best. Where the depth of water is not great and where the river is not subject to considerable changes of level the method of using a temporary platform on piling is convenient. Caissons for abutments and buildings may usually be built directly on the ground near the site where they are to be sunk.

When built on ways the caisson sometimes has a false bottom fitted to it to reduce the depth of immersion, and a sufficient height of crib is constructed, preliminary to launching, to insure the top being well above water-level. After launching and towing to the site more crib is added, the false bottom removed and the caisson sunk to the river bed by placing concrete in the crib.

The launching ways used for the McKinley bridge over

the Mississippi River at St. Louis, Mo., consisted of a number of rows of piles capped with timbers running at right angles to the river and on a slope of $1\frac{3}{4}$ inches per foot. Each caisson was built on shoes extending the full width of the caisson, the long side of the caisson being parallel to the river, and each shoe rested on a cap timber on which it slid during launching. These shoes were spaced about 6 feet apart and were so made that they projected down over the sides of the caps. They were bolted to the latter on the land side of the caisson. The caisson was built with its bottom in a horizontal position by using wedges between the caisson and the shoes. The launching was started by simultaneously sawing through the shoes below the bolts, which thus allowed the caisson to slide into the water.

Fig. 105*a* shows the caisson for one of the piers of the Vancouver bridge, Vancouver, Wash., as it was being built on the launching ways. The general scheme was about the same as for the McKinley bridge caissons.

Where built on floats, either one or two pontoons may be used. Fig. 105*b* shows one of the caissons of the Willamette River bridge of the Northern Pacific Railroad as it was being built between two barges or pontoons. The caisson was held between the barges until a height of 20 feet had been built up, when long screws were attached and the caisson lowered into the water. Two heavy trusses, one at each end, tied the barges together to prevent any unequal motion of the latter by the waves. Another caisson for the same bridge was erected on two pontoons, and after building to a sufficient height the pontoons were scuttled by filling them with water, after which they were pulled out from under the caisson.

The 78 × 144-foot caisson of the Manhattan bridge was built in a pontoon or float, 84 feet wide and 150 feet long, which had vertical sides 8 feet high. The float was built of 3-inch planks bolted to vertical and horizontal timbers. It was built in two halves separated by a longitudinal joint along the center line. Blocking was set up on the floor timbers and on this the caisson was built, thus making the latter accessible from below. On

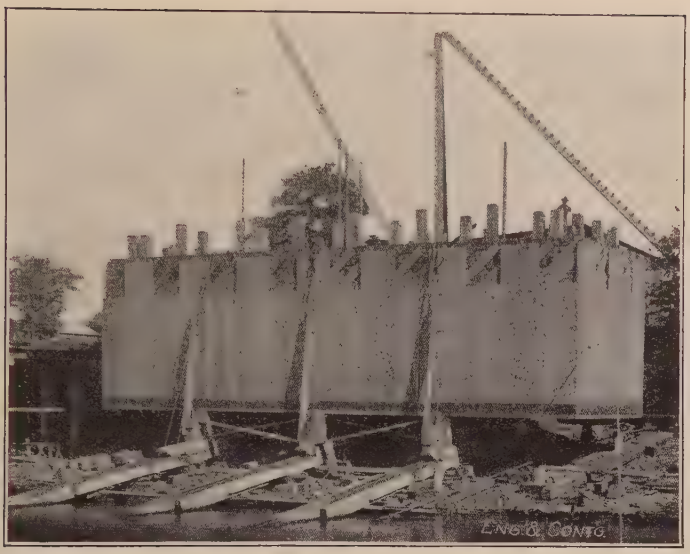


FIG. 105*a*.—Caisson on Launching Ways. Vancouver Bridge.

FIG. 105*b*.—Cassion Supported between Two Barges. Willamette River Bridge.

(*Facing p.* 316.)

FIG. 105*c*.—Launching a Caisson from the Pontoon in which it was Built. July 30, 1910. Municipal Bridge.

completing the caisson the joint between the two halves of the float was unlocked and sand dumped through the shafts of the caisson to the floor of the float to sink the halves of the latter, after which the same were pulled from beneath the caisson.

Fig. 105*c* shows one part of the 40×100-foot pontoon of the St. Louis Municipal bridge caissons as it was being pulled from beneath the caisson. This pontoon, which was of the same type as that described above, was sunk by removing plugs from holes in the bottom of the pontoon.

The caissons for the Passyunk Ave. bridge piers offer a good example of caissons built on a platform. Sixteen bearing piles were first driven in two longitudinal rows just clear of the caisson location. These were capped, and from these cap timbers four equidistant, transverse, 14×16-inch timbers were suspended by pairs of $1\frac{1}{2}$-inch rods, 16 feet long, threaded the entire length, and each provided with two nuts. Each transverse timber was held by means of a steel saddle on the under side, against which the lower nut of the rod bore and the other nut took bearing on a washer on top of the pile cap. The transverse timbers were first screwed up tightly against the under side of the cap timbers and on these the caisson was built. After building the cribs to a height of about 26 feet the caisson and transverse timbers were gradually lowered by unscrewing the nuts from the rods, which permitted the caisson to float in its exact position.

Art. 106. Sinking the Caisson

If mud covers the river bottom this should be dredged out before placing the caisson as it is cheaper to remove it in this manner than to excavate it within the working chamber. Great care must be exercised in grounding the caisson to place it in its correct position. If in tidal water, this may be done by placing concrete in the crib to an amount which will just ground the caisson at low tide. Then, by means of tackles attached to clusters of piles and to the caisson or crib, the structure is placed in its true position at high tide and grounded

as the water-level lowers. Concrete is then poured into the crib to an amount which will prevent floating when the tide rises. Often, where the caisson is slightly out of position, it may be floated by admitting a small amount of air into the working chamber. As soon as enough concrete has been placed to put on air pressure safely to expel the water from the working chamber, men enter to commence sinking operations.

In clay the excavation may usually be kept some distance below the cutting edge, which offers the advantage of allowing more head-room for the men. This cannot be safely done in sand as the water is very sensitive to changes of pressure and so it is not possible to raise the pressure very much from that corresponding to the head on the cutting edge. In one of the caissons of the Rulo bridge a test well was sunk in clay 17 feet below the cutting edge without any increase in the air pressure, but when a 4-foot vein of gravel was struck the pressure had to be increased 8 to 10 pounds at once.

In sinking caissons the load is at first usually carried on the cutting edge, but as the caisson gradually sinks more of the load is resisted by friction on the sides and less by bearing on the cutting edge. Contrary to the usual custom, in the case of the 55 × 180-foot caisson of the New Quebec bridge, the details of which are shown in Figs. 93*b*, *c*, and *d*, and which for the most part was sunk through sand, the load was not at any time supported on the cutting edge.

[1] "Owing to the great size of the caisson, extraordinary precautions were considered necessary to provide against any unequal settlement, or any twisting or other movement of the caisson, which might tend to open up the joints and seams and consequently allow air to escape. On this account it was decided that the ordinary method of sinking, where all the load is carred on the cutting edge, would not allow the movements of the caisson to be sufficiently controlled during the actual sinking. The rather unusual method was therefore employed of carrying the entire load on the bulkheads and the roof, and no load at all on the cutting edge.

[1] Engineering News, vol. 68, page 854, Nov. 7, 1912.

"The caisson was supported on 40 sand jacks, about 25 posts of 12×12-inch yellow pine, and 54 sets of blocking. The jacks and posts bore directly against the roof, while the blocking was piled under the bulkheads. When ready for a drop the blocking and posts were first removed by washing the sand from under them with a water-jet; then the whole caisson was lowered by operating all the sand jacks simultaneously. The sand jacks were of simple construction, each one consisting of a 29-inch steel cylinder closed at the bottom, having near the bottom two 3-inch holes with a sliding cover, and a plunger consisting of a single piece of timber fitting easily into the cylinder. The cylinder was filled two-thirds full of sand, the plunger inserted, and its upper end blocked against the roof. The operation of lowering consisted in opening the lower holes and inserting a water-jet, thus washing out the sand.

"These jacks worked admirably, the result being that the caisson was sunk absolutely level and in its proper location. Before each drop a trench was excavated under the cutting edge to a depth of 2 or 3 feet, and filled with clay, which tended to prevent the escape of the air and also acted as a lubricant during sinking. This scheme was followed throughout the entire sinking and seemed to materially facilitate the operation."

Sinking the caisson is accomplished by excavating the material in the working chamber and by placing concrete in the crib to weight the structure. The water-jet is sometimes employed to reduce friction on the sides.

ART. 107. REMOVING SPOIL FROM WORKING CHAMBER

Various devices have been developed for removing the spoil from the air chamber. Where the material is sand the blow-out process or mud-and-sand pump is ordinarily employed; where clay is encountered it is usually best to remove it with buckets, using some simple form of air-lock, or perhaps the clay may be mixed with water and the sand-and-mud-pump process used. Boulders must be removed through the air-locks.

BLOW-OUT PROCESS.—The blow-out process is a very simple

affair, the principle consisting of using the pressure in the air chamber to drive out sand or mud when it is piled around the inlet of a pipe which leads from the working chamber to the open air. The diameter of the pipe is usually about 4 or 5 inches, the top being fitted with an elbow to throw the sand in a horizontal direction, while the lower part has attached to it a flexible hose of large diameter with a valve. To blow out the sand and mud it is only necessary to heap it up around the mouth, open the valve, and the material is then carried out with a high velocity; in fact the velocity is so great that the pipe rapidly wears away. At the Havre de Grace bridge the elbow, which was of chilled iron, 4 inches thick, was worn through in two days. Considerable care must be exercised in placing the material against the inlet for if a considerable amount of air is not admitted with the sand and mud, it will clog, while if there is too much air admitted it is a waste. It has been found advantageous to have small holes in the pipe above the inlet as this gives more uniform action, tending to draw the material up instead of merely driving it and thus lessening the amount of air entering with the sand and mud. Although the dry blow-out is a very rapid and satisfactory means of removing the spoil from the working chamber it has some disadvantages: First, a tendency to vary the pressure in the working chamber; and second, a tendency to cause rapid wear in the pipe elbow as noted above. The lowering of the pressure due to the air passing up through the pipe causes a very thick fog, making it difficult for the workmen to see. It is also apt to allow the water to enter from the outside. On the other hand, if the air compressors are supplying air at a rate sufficient to maintain a constant air pressure when the sand is being blown out, on stopping the latter operation the pressure may rise to a point sufficient to cause a blow-out under the cutting edge, which is usually followed by a flooding of the air chamber. Largely on account of the destructive action on the pipe, and for the added reasons just noted, the dry blow-out process is most satisfactory when the pressure in the working chamber is fairly low, although a head of at least 20 feet is necessary. This process is said to

have been used first by WILLIAM SOOYSMITH in 1859 in building bridge piers over the Savannah River.

SAND-AND-MUD PUMP.—The principle involved in this form of excavator is that of the induced current, where a quantity of water with a high velocity causes a reduction of pressure which draws the mud and sand—well mixed with water—into the pipe. Fig. 107 *a* illustrates the form used on the Memphis bridge. The water enters at the side under a high pressure and passes up through the small annular space, at which point, on account of the high velocity, the pressure is low. The lower part of the pump connects with a pipe or hose, the lower end of which rests in a pool of mud or sand and water. On account of the difference of pressure at the two ends of this pipe the mud is drawn into the pump and carried upward with the water, through a pipe which connects with the top of the pump. The essential difference between this form of excavator and the blow-out process is that in the former the water is the moving force doing the work while in the latter it is the air from the working chamber. The water pressure used is ordinarily about 80 pounds per square inch. This method was first used by JAMES B. EADS in the caissons of the St. Louis arch bridge. Fig. 91*a* illustrates another form of the sand-and-mud pump.

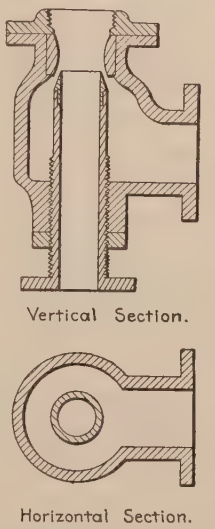


FIG. 107*a*.—Sand-and-Mud Pump. Memphis Bridge.

In the Williamsburgh bridge, New York, the hose was extended to a sort of sump in the bottom of the excavation where its open end was placed below the surface of the water. Gravel, sand and mud were constantly fed into the nozzle by a laborer who raked it up and prevented clogging, and another man with a $\frac{3}{4}$-inch nozzle played a 50-pound water-jet against the soil to wash it into the sump.

For a description of this process as applied to open-caisson work the reader is referred to Art. 91. In some caisson work at

Arran, Switzerland, instead of using a sump a horizontal hopper was employed, the discharge pipe leading from the lowest point in the hopper. A jet of water from a small pipe was constantly played on the material as it was fed into the hopper.

REMOVING MATERIAL WITH BUCKETS.—Clay is usually more cheaply removed with buckets than by any other method. Large rocks must be blasted to pieces and removed with buckets. As stated in Art. 103 where a form of lock similar to the Moran or O'Rourke lock is used, the bucket may be taken from the lock without removing it from the hoisting rope. In the form shown in Fig. 103*a*, instead of running the hoisting rope to an engine on the outside, the hoisting is done by compressed air from the working chamber working in the cylinder shown on the left. In this cylinder runs a piston, the two sets of sheaves being so arranged that one stroke of the piston lifts the bucket the whole distance.

A novel device, called the water column, was used in the caissons of the Brooklyn bridge to remove the material. It consisted of an open shaft, the lower part extending into a sump which was kept full of water and the shaft itself was filled with water up to a point sufficient to balance the air pressure in the caisson. Workmen pushed the spoil under the shaft and from there it was removed by dredging with an orange-peel or clam-shell bucket.

ART. 108. CONCRETING THE AIR CHAMBER

When rock is reached, if the same is level, it is only necessary to clean off all loose material before depositing the concrete. On the other hand, if not level, some preliminary work must be done; if the rock has a uniform slope it should either be blasted down to a level surface or else stepped, unless very rough; although if the rock surface is at practically the same elevation all around the cutting edge of the caisson, but irregular within, little more than a thorough cleaning will be necessary. For those caissons founded on clay or hard-pan a level surface is easily obtained.

Caisson No. 10 of the Passyunk Ave. bridge landed on rock which had a slope of about 5 feet in the length of the caisson. As soon as rock on the high side was reached, the cutting edge on the low side was blocked with 6×12-inch timbers, 6 feet apart, after which excavation under the cutting edge was carried to rock and extended $1\frac{1}{2}$ feet out beyond the cutting edge. This excavation was then filled with concrete.

In the caissons for the St. Louis Municipal bridge the rock surface was irregular but no attempt was made to level it off or to bring the caissons to bearing throughout. Where depressions occurred the sand was removed and sacks of concrete were deposited on the rock and tamped under the cutting edge, after which concrete was placed in the working chamber in the usual manner.

The concrete for filling the working chamber may be carried in through the material shafts and locks by means of buckets, or special arrangements may be made, by placing a cone-shaped frame above the lower door, by which a yard or more of concrete may be dumped into the lock through the upper door. The latter is then closed and air admitted to the lock allowing the lower door to open and the mass of concrete to fall through the shaft to the working chamber. The conical frame prevents the concrete from remaining in the lock when the lower door is opened. For a description of the method used in placing the concrete in the working chamber see Art. 186.

ART. 109. RATE OF SINKING

The rate of caisson sinking varies greatly, the larger the caisson and the harder the material sunk through, the slower the rate. Sinking operations are usually carried on day and night, and the rate of sinking will vary from almost nothing where beds of boulders are encountered to as much as 3 feet a day where clean sand is met. Most engineers keep a chart of the progress of the work; Fig. 109*a*, which illustrates the progress in sinking one of the caissons of the Kinzie St. draw-bridge, Chicago, is a very satisfactory form of chart to use. The

caisson is shown in Figs. 109*b* and *c*. Instead of carrying the whole caisson to bedrock the cutting edge was stopped about half way down and wells were then sunk the remainder of the distance.

In sinking Pier *D* of the Memphis bridge, excluding long delays, an average rate of 1.5 feet per day of 24 hours

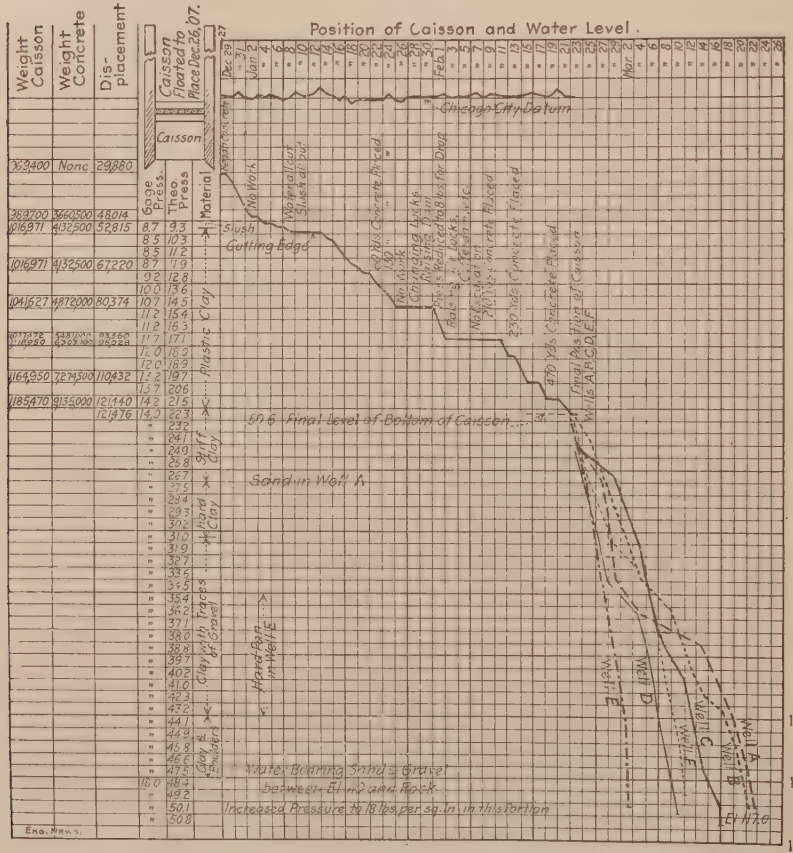


FIG. 109*a*.—Progress of Sinking Caissons. Kinzie St. Bridge, Chicago.

was maintained through sand and only 0.31 foot through clay, while for Piers 2 and 5 of the Thebes bridge the average rates were 0.23 and 0.41 foot respectively; here hard gravel was encountered.

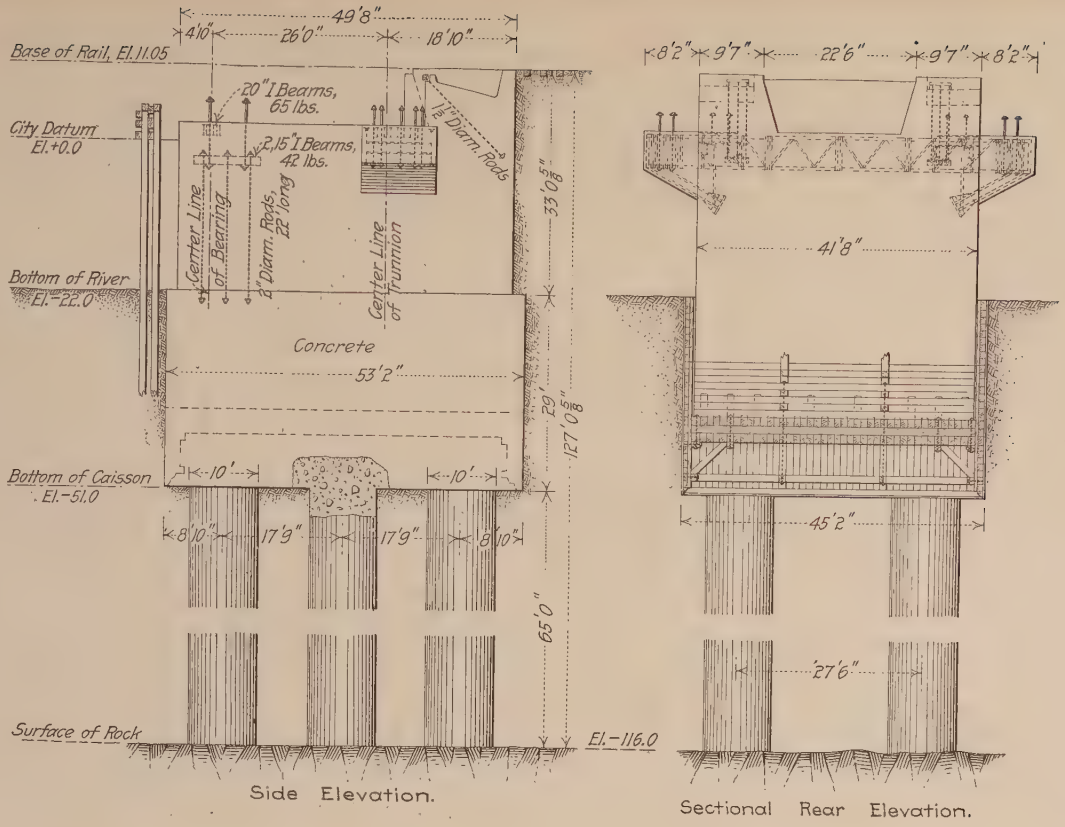


FIG. 109b.—Pneumatic Foundations for C. & N. W. Ry. Bridge, Kinzie St., Chicago.

The rate per day of sinking the St. Louis Municipal bridge caissons varied from an average of 0.68 foot for Pier 4 to 1.95 feet for Pier 3, with 1.28 feet as an average for all caissons. The best progress in one day was 5.17 feet, while the best seven-day run was 34 feet or 4.86 feet per day. For the caissons of the McKinley bridge, St. Louis, the average rate for all caissons was 2 feet per day, with a maximum of 7.7 feet in one day.

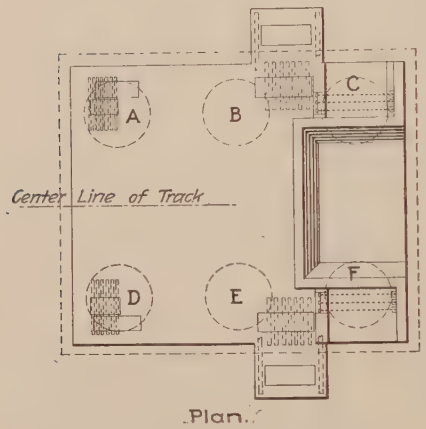


FIG. 109c.—(See also Fig. 109b.)

ART. 110. FRICTIONAL RESISTANCE

Estimating the probable frictional resistance to be met with in sinking caissons is one of the most difficult features involved in the design. It depends upon numerous factors such as the kind of material penetrated; the material composing the sides of caisson and crib; depth to which sunk; whether the sides of the caisson are vertical or flared; whether or not the water-jet is used; and the amount of air leaking under the cutting edge.

In general, the frictional resistance per square foot of exposed surface of caisson and crib will seldom be less than 250 nor more than 800 pounds, although in boulder-strewn material it may be as much as 1000 pounds. Next to mud and silt, sandy soils offer the least resistance, especially when carrying large amounts of water, while clay will offer less resistance than material containing boulders. With uniform soil conditions the unit friction will increase with the depth; for instance, at the McKinley bridge, which crosses the Mississippi River at St. Louis, the friction was found to be about 300 pounds per square foot of exposed surface at 40 feet, and 600 pounds at a penetration of 70 feet. Anything which tends to loosen the soil around the

sides of the caisson and crib will decrease the friction, at least for a short time; escaping air has about the same effect as the water-jet in lubricating the material. Although flaring out the bottom of the caisson tends to reduce the side friction, yet, on account of possible wedging action by material falling into the open space above the bottom, and further, on account of the loss of guidance, pneumatic caissons are now practically all made with vertical outside walls.

Table No. 110*a* gives values for the skin friction when the caissons were well down for a number of notable structures. Table No. 110*b*, taken from an article by H. L. WILEY in Transactions American Society of Civil Engineers, vol. 62, page 113, March, 1909, gives values of friction for both open and pneumatic caissons.

TABLE NO. 110*a*

SKIN FRICTION FOR PNEUMATIC CAISSONS OF BRIDGES

(Expressed in Pounds per Square Foot)

Name of bridge	Range for separate piers	Average	No. of piers	Materials penetrated in sinking caissons
Bellefontaine	600–700	648	4	Fine sand, sand, coarse sand, boulders.
Blair Crossing	330–410	381	4	Fine sand, coarse sand, clay.
Brooklyn	600			
Cairo	622–932	750	10	Sand.
Havre de Grace	308–489	400	4	Silt, sand, mud.
McKinley	600			
Memphis	365–837	584	5	Sand, gravel, mud, clay, sediment, very tough clay, quicksand.
Miles Glacier	620			
Nebraska City	409–590	525	3	
New Omaha	472–673	617	5	Sand, gravel, some clay to bedrock.
Rulo	351–944	614	4	River sand, coarse sand, rubbish, clay, gravel.
Sioux City	314–535	463	4	Fine sand, yellow sand, gravel, clay, boulders.
Williamsburg	750			

General average for nine bridges, 554 pounds per square inch.

TABLE NO. 110*b*

No.	Type of caisson	Method of sinking	Material penetrated	Skin friction	Depth below low water in feet	Area of base in square feet
1	Cast iron	Open excavation	Gravel, clay	240	60	125
2	Cast iron	Open excavation	Sand, clay	250	75	225
3	Cast iron	Open excavation	Sand	250	60	125
4	Wrought iron	Open excavation	Sand, clay	285	140	1000
5	Cast iron	Open excavation	Sand, clay, gravel	300	100	125
6	Cast iron	Open excavation	Sand	325	60	125
7	Cast iron	Open excavation	Silt	350	60	125
8	Steel construction	Open excavation	Silt, sand, clay	375	55	190
9	Cast iron	Open excavation	Silt, mud, clay	390	75	100
10	Timber construction	Open excavation	Sand	450	30	1300
11	Steel construction	Open excavation	Silt, clay	450	60	700
12	Steel construction	Open excavation	Silt, clay, sand	450	60	1200
13	Steel construction	Open excavation	Mud, sand	450	65	1300
14	Steel construction	Open excavation	Clay	450	75	1500
15	Iron construction	Open excavation	Sand, gravel, clay	480	65	200
16	Cast iron	Open excavation	Clay	500	60	125
17	Steel construction	Open excavation	Clay	700	65	1300
18	Masonry	Pneumatic	Sand, mud	265	40	75
19	Timber construction	Pneumatic	Clay	250	35	800
20	Steel construction	Pneumatic	Clay, sand	275	60	150
21	Timber construction	Pneumatic	Silt, sand, mud	310	75	2550
22	Timber construction	Pneumatic	Sand, clay, gravel	350	100	1200
23	Timber construction	Pneumatic	Sand, clay, boulders	400	48	1925
24	Timber construction	Pneumatic	Clay, sand, gravel	400	95	4500
25	Timber construction	Pneumatic	Sand, gravel, clay	425	55	1300
26	Steel construction	Pneumatic	Sand, boulders	450	68	2700
27	Timber	Pneumatic	Silt, clay, gravel	500	75	1800
28	Iron cylinder	Pneumatic	Sand, shale	525	60	1200
29	Timber construction	Pneumatic	Sand	540	75	1700
30	Timber construction	Pneumatic	Sand, clay	600	75	1400
31	Timber construction	Pneumatic	Sand, gravel, clay	650	80	2000
32	Timber construction	Pneumatic	Sand	650	90	1200
33	Timber construction	Pneumatic	Sand, boulders	660	101	2100
34	Timber construction	Pneumatic	Silt, sand, clay	900	45	1700

In sinking the Commercial Cable Building caissons the frictional resistance varied from 250 to 300 pounds per square foot of exposed surface, while in the United Fire Insurance Co. caissons it was as high as 1000 pounds.

The highest value of frictional resistance was observed in 1910 while sinking the concrete caisson for the pivot pier of the reconstructed swing bridge of the Grand Trunk Railway at Black Rock Harbor on the Niagara River. The material penetrated was a very sticky red clay. The concrete open caisson weighed 8700 tons and 1084 tons of stone and pig iron were

piled on top of it. The area was 10235 square feet, thus giving a frictional resistance of 1912 pounds per square foot.

Art. 111. Physiological Effects of Compressed Air

The question of the physiological effects on the human system when working in compressed air is an important one from both the humanitarian and financial standpoints. In the past almost all the important works employing compressed air have levied a heavy toll of suffering and death on the 'sand-hogs,' as caisson workers are commonly called. For instance, on the caisson work of the St. Louis bridge there were 119 cases of so-called caisson disease, with 14 deaths from the same, while on the Brooklyn bridge there were 110 cases of illness, with 3 deaths. These, of course, were early examples; at the present time, owing to a better knowledge of the disease, the records are not so bad, but the disease still claims its victims in too many cases.

No harmful effects are felt on entering the compressed air, or while remaining in it; only during decompression or after emerging are the workmen affected. The disease, which has been proven to be aeremia, may be divided into two classes: First, that in which the attack is light; and second, that in which it is severe. The first form is characterized by very severe pains, chiefly in the joints, and closely resembles rheumatism in its effects. From the tendency to cause its victim to double up in agony it is commonly known as the 'bends.' When the attack is very severe it usually paralyzes its victim and is commonly fatal.

Sensations Felt on Entering the Air Chamber.—On entering the air-lock and having the air pressure turned on, some of the sensations felt are heat, slight giddiness and headache, pain in the ears, breathlessness, inability to whisper—caused by the resistance of the compressed air to the finer muscular movements of the tongue—and a feeling of resistance to movement owing to the density of the air. A slight discomfort is usually felt in maintaining equilibrium between

the air pressure inside and outside the body, the most painful being in the ears, as noted above. This may be overcome by closing the mouth and holding the nose, and at the same time trying to expel the air from the lungs; such action makes the pressure in the tympanic cavity equal to the outside pressure by means of the Eustachian tubes which run from the back of the nasal passages to the cavity. This action should be repeated from time to time and as long as the pressure continues to increase. Relief may also be secured by the action of swallowing. A cold makes the feat more difficult since the Eustachian tubes are then somewhat blocked.

Owing to breathing the denser air with its increased amount of oxygen, as soon as equilibrium obtains the general effect is somewhat exhilarating and bracing. To quote one of the workmen in the Blackwall tunnel (England): [1] "I never felt happier than when I was in the compressed air. Always happy, and on the cheery side. Why, laddie, I would get up in the morning feeling very dour and queer, and just go into the workings and then whistle (?) and sing all day long."

SENSATIONS FELT ON LEAVING AIR CHAMBER.—On leaving the air pressure the caissonier feels cold, and this is felt most keenly during the passage through the air-lock, being due to the expansion of the air in the lock, as well as to the expansion and liberation of gases in the body. To counteract the effects of this cold the air-lock should be warmed and the men given strong hot coffee to drink on emerging, and should dress warmly. Another characteristic of decompression is a dense fog which occurs as the air becomes rarefied. Another sensation often manifested on emerging is an itching, pricking feeling under the skin on all parts of the body; this disappears in a few minutes. The foregoing are the sensations always felt; if the person is taken with caisson illness the symptoms may be manifold.

[1] "Coming out again (from the working pressure) it was not so bad, but just chilly; bitter chilly, cold as charity. The pains would come on afterward, in an hour or so, or when you got into bed. Bends in the

[1] Engineering News, vol. 51, page 437, May 5, 1904.

back, the wrists and the legs; just awful. Men would turn out in the middle of the night and come back to the works and get into the compressed air again in the medical locks. They had a full dose for a start, and let the pressure drop gradually. Then they went back home to bed. Do them any good? Eh, mon, its no for me to say. They thought so, but I thought it was only humbug, a faith dodge. When I had bends I just jumped about and took a drap of guid whuskey—better than all your doctor's concoctions." The foregoing graphic description of the 'bends' and treatment for it indicates the attitude of the average 'ground-hog.'

[1] "The symptoms of caisson disease have been quite definitely established. First among these are neuralgic pains of an intermittent or paroxysmal character, and of varying severity. In the worst instances these pains, or cramps, as they are commonly called—although they are rarely accompanied by muscular spasms—are so intense as to completely unnerve strong men. This symptom is very seldom absent, and from it comes the popular name of 'bends' given to the disease. Another characteristic symptom which is always exhibited is a profuse cold perspiration. Another symptom which is of frequent occurrence, but which is not always exhibited, is pain at the pit of the stomach, usually, but not always, attended by vomiting. In about 50 percent of the cases observed, paralysis has been a characteristic symptom. The degree of paralysis varies from slightly impaired sensation or numbness in the extremities to complete loss of sensation and motion in the affected parts, which are most frequently the legs and lower part of the body. Finally the sufferer usually exhibits a number of transient symptoms, which have their origin in the brain; these are headache, dizziness, double vision, incoherence of speech, and sometimes unconsciousness. The duration of these symptoms varies from a few hours to several weeks in case of paralysis. In fatal cases congestion of the brain or spinal cord always exists. A very noticeable fact is that the attack of the disease never takes place while the subject is under air pressure, but always occurs while he is emerging from the compressed air chamber or after he has emerged."

CAUSES OF CAISSON DISEASE.—Various theories have been advanced from time to time relative to the cause of caisson disease. It is said that attention was first called to caisson disease at about the middle of the last century by TRIGER who applied the use of compressed air in sinking some coal shafts at Chalons on the banks of the Loire. [2] "HOPPE SEYLER (1857) and THOMAS SCHWANN (1858) in Germany, and BUSQUOY (1861)

[1] Engineering News, vol. 46, page 157, Sept. 5, 1901.
[2] Engineering Record, vol. 63, page 362, April 1, 1911.

in France, . . . gave the first correct suggestion as to the cause: viz., that it was due to the setting free of bubbles of gas in the blood. Nitrogen gas is dissolved, according to the law of partial pressures, during exposure to the compressed air, and this dissolved gas having no time to escape through the lungs, if the pressure be suddenly lowered, bubbles off just as carbonic acid escapes from aerated water when a bottle is uncorked."

In 1871, DR. JAMINET, the physician in charge of the compressed air workers at the St. Louis bridge, became convinced from his studies that the disease was caused by too rapid a tissue change due to the absorption of an excess of oxygen.

About two years later, DR. A. H. SMITH, the surgeon in charge of the New York tower caisson of the Brooklyn bridge, arrived at the conclusion that the ill effects developed in working under compressed air were due to the pressure of the air forcing the blood from the surface of the body to the center and thereby causing internal congestion.

But it was PAUL BERT, who, by his remarkable experiments, published in 1878, proved the true cause of caisson disease to be the effervescence of gas in the blood and tissue juices. Since then such authorities as PHILLOPON, VON SCHROTTER, HELLER, MAGER, HALDANE, HILL, SMITH, MACLEOD, GREENWOOD and others, have checked and extended BERT'S experiments.

The gas which is present in the blood, and which comes out of solution if the pressure is too rapidly lowered, is mostly nitrogen, for if the chamber is properly ventilated there will be only a small amount of carbonic acid gas in the air, while the oxygen content dissolved by the blood is taken up chemically by the hemoglobin, as demonstrated by DR. HALDANE. As stated elsewhere the tissue fluids, chiefly the blood, dissolve the air according to DALTON'S law of solution of gases in fluids; *i.e.*, the amount of gas dissolved in a fluid is proportional to the pressure of the gas surrounding the fluid. Except for very high pressures, such as eight or ten atmospheres—values which will never attain in caisson work—these dissolved gases probably have no chemical effect on the system, and are quite harmless as long as they remain in solution. For high pressures the dis-

solved oxygen seems to have a toxic effect, causing a fatal inflammation of the lungs. Experiments have shown that with a pressure of ten atmospheres some animals will die in as short a time as 20 minutes.

However, when the pressure of the surrounding air is lowered, the dissolved gases, mostly nitrogen, are thrown out of solution in the form of bubbles. If the lowering of the pressure is done slowly the gases are thrown out of the blood at the lungs without developing bubbles of any appreciable size. But if the pressure is rapidly lowered the gas bubbles stick, owing to their size, in the minute blood vessels and obstruct the flow of the blood, often causing the vessels to burst. The same condition obtains in the various tissues carrying juices saturated with gas; if these bubbles develop in the joints, we have the 'bends'; if in the spinal cord, paralysis; if in the heart, heart failure, etc.

ART. 112. PREVENTION OF CAISSON DISEASE

If the cause of caisson illness is a mechanical action due to the development of bubbles in the blood and fluid tissues, which in turn is due to too rapid decompression, then manifestly the cure is decompression at a rate slow enough to avoid this phenomenon. The length of time will depend upon the amount of gas in the fluid tissues and upon the physical characteristics of the person being decompressed. The amount of gas in the fluid tissues will, in turn, depend upon (1) the degree of pressure in the working chamber and (2) the length of time under pressure. The length of time taken to saturate the body fluids at any particular pressure will vary greatly, depending upon the fatness of the subject, the amount of bodily work done, heat and moisture present, etc. From experiments DR. HALDANE concluded that in certain parts of the body where the circulation is rapid and the number of blood vessels high the tissue juices will become 50 percent saturated in 5 minutes, with complete saturation in 40 minutes; while other parts, lacking a copious supply of blood, will require 75 minutes for 50 percent saturation

and about 4 hours for 90 percent saturation. Experiments show that the fatty tissues absorb about five times as much gas as does the blood and the rate of absorption is much slower; the rate of desaturation will be correspondingly slow. For this reason men inclined toward fatness should never be employed for compressed-air work. The better the circulation of the blood the more quickly and easily will the gases be thrown out of the system; for this reason only men in good physical condition should be employed. Old men, or those who have abused themselves by excessive drinking or other dissipation, should never be allowed in the working chamber.

Authorities differ as to the time that should be allowed for decompression, but all agree that the usual time given is too short. Some urge a uniform rate of decompression, while others prefer stage decompression, that is, at first a rapid decompression to a certain pressure, followed by slower decompression.

Seldom is more than 15 or 18 minutes given to decompression; the reason for this is that the air-lock is small and as a consequence the men must maintain cramped positions in the same. Moreover, the lock is usually cold and filled with fog, due to the decreasing pressure. Properly, the lock should be large enough to allow the men some freedom of motion and it should be ventilated with warm dry air. The French law, enacted in 1908, prescribes that for a head of water up to 65.6 feet not less than 21.2 cubic feet of air shall be provided for each man in the lock, and for depths above this not less than 24.7 cubic feet. During decompression the men should constantly move about and massage their various joints, as this has been found to assist materially in ridding the system of the gases.

MACLEOD suggests the following time for decompression as being safe:

Gage Pressure	Length of Shift	Decompression Period
15 to 30	4 hours	½ to 1 hour
45 to 60	4 hours	1½ to 2 hours

In Germany, VON SCHROTTER, HELLER and MAGER, in 1900, published a work in which they laid down the principle that a

uniform decompression at the rate of two minutes per 0.1 atmosphere, or 20 minutes per atmosphere, was safe.

The law of New York State (1913) governing the time of decompression for pneumatic caisson work for bridges and buildings is as follows:

Gage pressure in pounds	10	15	20	25	30	36	40	50
Time of decompression in minutes	1	2	5	10	12	15	20	25

The time of work in caissons, given by this law, is as follows:

Gage pressure	0–21	22–30	31–35	36–40	41–45	45–50
Time per day in caisson.	8 hrs.	6 hrs.	4 hrs.	3 hrs.	2 hrs.	1½ hrs.
No. of shifts.....	2 (minimum)	2	2	2 (min.)	2 (min.)	2
Length of shift....	.....	3 hrs.	2 hrs.	1½ hrs. (max.)	1 hr. (max.)	¾ hr.
Minimum time between shifts.	30 consecutive minutes	1 hr.	2 hrs.	3 hrs.	4 hrs.	5 hrs.

The theory upon which stage compression is based is that the gas in the blood will not effervesce until a marked diminution of pressure obtains, and as, to the point of effervescence, the gases are discharged at a rate varying with some function of the change of pressure, manifestly the more rapid the lowering of pressure the more quickly will the blood vessels be freed of the gases contained therein. Since almost no cases of aeremia are caused by rapid decompression from about 19 pounds gage pressure, it seems reasonable to assume that the pressure in the air-lock may be reduced that amount in about three minutes; from this point the pressure must be lowered quite slowly and should correspond to the natural rate of desaturation of the fluid tissues at that difference of pressure. When the gage pressure reaches about 19 pounds, the remainder of the decompression may be done quickly, for, as stated above, it appears that the average person can safely stand that difference of pressure. The fundamental idea upon which stage decompression is based is correct, but as there is but little experimental data and less precedent to guide one, it has not yet become general.

Apart from the matter of slow decompression, other precau-

tions, if taken, will do much to lessen the occurrence of caisson disease. Anything which tends to lower the vital resistance of the human system tends to promote caisson illness. For this reason the physical conditions under which the men work should be as good as it is possible to make them: There should be furnished plenty of fresh air; electric lighting rather than gas or candle lighting should always be employed, as the latter tends to vitiate the air; the air should be kept at as reasonable a temperature as possible, which means that it should be cooled during the summer time, as compression raises its temperature. At present this is done in practically all work, either by spraying the compressed air as it enters the working chamber, or else by passing it through a coil of pipes externally cooled.

[1]"It is well known that, in a confined atmosphere, man sooner or later suffers from the accumulation of poisonous gases. The criterion of this pollution of the atmosphere is the amount of carbonic acid (CO_2) found present. When the percentage of CO_2 in the air rises above 0.1 percent, evil effects are common. It should be clearly understood that these evil effects are not due to the carbonic acid itself, but to some other toxic property which the CO_2 content seems to run parallel with, and is, therefore, a measure of it. Now under pressure it is evident that such a gas will be still more dangerous. As a matter of fact, E. H. SNELL reports that an 'increase of CO_2 from 0.04 percent to 0.1 percent at 30 pounds pressure is the forerunner of much illness.' He found that by free ventilation of the caisson, so as to remove this CO_2, the illness dropped from seven cases a day to one case in two days. . . . Ventilation is a matter which should be carefully provided for, since otherwise the CO_2 and other poisonous constituents of polluted air will have their usual depressing effects on the workmen and render them more prone to suffer from decompression symptoms."

Especially when sinking through foul material should care be exercised in keeping the air pure. T K. THOMSON reports that when sinking through the foul bottom of the Harlem River the

[1] Cause, Treatment and Prevention of the Bends, by J. J. R. MACLEOD, Journ. Assoc. Eng. Soc., vol. 39, page 301, Nov., 1907.

men suffered much from the bends, but when sinking through the clay below this, even though under a much greater pressure, very little trouble occurred. It is also noticed that a greater amount of sickness is apt to occur during concreting than at other times, this being due to the decrease in the leakage of the air, or inadequate ventilation.

CURE FOR CAISSON DISEASE.—The best and about the only cure for caisson disease is recompression with slow decompression. If the patient can be put into the air before the gas bubbles have had a chance to tear the blood vessels and fluid tissues a cure can usually be effected, but otherwise not. For this reason, a hospital air-lock, large and well ventilated, should always be maintained in readiness and the men should be housed near by, so that in case of delayed attacks they may be immediately recompressed.

CHAPTER X

PNEUMATIC CAISSONS FOR BUILDINGS

Art. 113. General Development

The application of the pneumatic caisson to building foundations has been restricted very largely to the tall buildings or 'skyscrapers' of New York City. Two conditions occur there which require this form of foundation: First, the necessity for carrying the column loads to bedrock; and second, the presence of quicksand over the rock. Both the height of the buildings and the magnitude of the column loads make it imperative to found the piers on a very hard and unyielding stratum, preferably bedrock, since any irregular settlement is exceedingly dangerous and difficult to remedy in tall buildings. The presence of quicksand makes sinking to bedrock very difficult by other methods than that of the pneumatic caisson, due to the tendency of the material to flow into the excavation; while it is especially dangerous in the lower part of Manhattan Island, due to the liability of undermining adjacent building foundations, many of which rest on shallow foundations. The only disadvantage of the pneumatic method is its high cost, but this is fully justified where the security of very expensive buildings is at stake.

In its details, the caisson for a building does not differ materially, except in the matter of size, from the bridge caisson. It is customary in most cases to use separate piers for all the interior columns, these being circular or square in plan; but special conditions, such as the close spacing of two or more columns, or lack of clearance, sometimes makes it necessary to use one pier for two or more columns. Where the grade of the cellar floor is below the ground-water line the wall piers often serve two functions: First, that of carrying the wall-column loads to rock;

thus making the beam spacing 8 feet center to center, the shaft spacing longitudinally being 16 feet center to center.

For valuable material on the design and costs of various types of abutments see a paper by J. H. Prior in Proceedings of American Railway Engineering Association (1912), vol. 13, page 1085, as well as an article by W. M. Torrance on The Design of High Abutments, in Engineering News, vol. 55, page 36, Jan. 11, 1906.

CHAPTER XV

SPREAD FOUNDATIONS

Art. 149. General Considerations

Foundations for buildings, where bedrock is some distance below the surface, are of three general types: First, those carried deep to rock or hard-pan; second, those in which piles are used; and third, those spread over a given surface. The first type is widely used for heavy buildings where the material overlying the rock is soft, and is exemplified in the pneumatic-caisson process described in Chap. X, and in the open-well process described in Chap. XI. Although the most expensive type of foundation, it offers the advantage of an absolutely unyielding support for the buildings. The subject of bearing piles is treated in Chaps. I to V inclusive.

The object of the shallow type of foundation is to spread the load over a considerable horizontal area near the surface of the ground; that of pile foundations to distribute the load over a considerable vertical area—the circumferential surface of the piles—as well as carrying some of it to the horizontal stratum at the feet of the piles; while the deep foundation distributes the load over a relatively small area on the rock or hard-pan. Where rock is present near the surface there is no foundation problem, it being necessary only to level off the rock with a layer of concrete and place the columns or walls directly upon it, although a spread footing may be used where the foundation loads are very heavy.

In many localities the most common type for light buildings is the shallow foundation, and in modern development it is being used to a considerable extent for heavy structures. In its original and simplest form the shallow foundation consists of a wide concrete or masonry footing with its maximum area at

the base and stepped off to decrease in horizontal area toward the top, the latter being of sufficient size to form a seat for the wall or column base. Although this makes a satisfactory footing for small loads it is not well adapted to heavy loads owing to the depth required to get the necessary spread of base. Other forms of shallow foundations have been developed, such as the wooden grillage, the inverted arch, the steel I-beam grillage, and the reinforced-concrete spread footing, all of which require less depth.

The shallow type of foundation is relatively inexpensive, and easily and quickly constructed, but it possesses the disadvantage of failing to furnish a rigid and unyielding support for the building. Where founded on compact sand the settlement will be slight, seldom more than $\frac{1}{2}$ inch, but where founded on a material like the Chicago clay the settlement may in time amount to 2 feet or more. Hence heavy buildings resting on shallow foundations are built to allow for a certain amount of settlement, or else the foundations are so constructed that powerful hydraulic jacks can be used to raise the building to permit shimming up. Uniform settlement causes but little trouble and can be easily taken care of; but unequal settlement causes the walls to crack. The most satisfactory method of guarding against unequal settlement was early found to be the use of independent footings for the columns, the area of the base of each footing being so proportioned that the unit-pressure is the same under all footings.

ART. 150. EARLY TYPES OF FOOTINGS

MASONRY FOOTINGS.—This type, which was one of the earliest, is still the standard for light loads. It may be built of concrete, brick masonry, or stone masonry, the first being the most widely used at present. In designing the footing the area of base is found by dividing the wall load by the safe bearing power of the soil as given in Art. 179. To safeguard the masonry against crushing the compressive unit-stress on any horizontal section should not exceed the values given in Table

150*a*. The top of the footing is made a little larger than the column base or wall.

Having determined the top and bottom areas of the footing the next step is to design the offsets, which fix the depth of the footing. As usually designed these offsets are assumed to act as free cantilevers, and so the allowable offset of any section will depend upon: First, the pressure on the under side; second, the transverse strength of the masonry; and third, the thickness of the course. The center of gravity of the base should coincide with the axis of the load, otherwise additional stresses will develop.

TABLE 150*a*

Character of masonry	Safe compression, lbs. per sq. inch
Common brick, hard burned (portland cement mortar)	200
Common brick, ordinary (portland cement mortar)	175
Rubble masonry, uncoursed (portland cement mortar)	140
Rubble masonry, coursed (portland cement mortar)	200
Portland cement concrete, 1–2–4 mixture	450
Portland cement concrete, 1–2½–5 mixture	350
Portland cement concrete, 1–3–6 mixture	250

Considering the case of a footing for the wall of a building, let p denote the unit-pressure in pounds per square foot on the bottom of the course in question; R, the modulus of rupture of the masonry; f, the factor of safety used; t, the thickness of the course in inches; and o, the allowable offset of the course in inches. The following formula is then obtained,

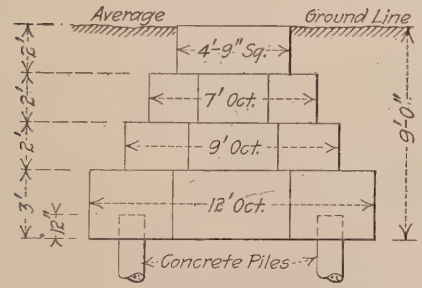
$$o = t\sqrt{48R/(pf)}.$$

A factor of safety of about six will usually be advisable. In designing masonry footings for columns the method given in Art. 158 is recommended, although the above formula may give sufficient precision.

OTHER EARLY TYPES OF FOOTINGS.—Owing to its lack of transverse strength masonry is ill-adapted to take loads which cause flexural stresses of any magnitude. For this reason vari-

ous substitutes have been adopted, the idea being to use some material having considerable transverse strength in order to reduce the necessary depth.

Among the early types was the timber grillage. This consists of two or more layers of heavy timbers, each layer being placed at right angles to the one above and below, the top and bottom being often sheathed with a layer of planking. The various courses are well tied together with drift bolts. Examples of such grillages have been dug up after being buried from 50 to 100 years and where below ground-water level have been found to be in a perfect state of



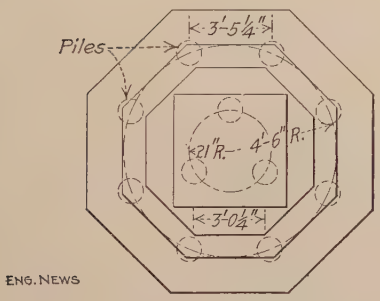


FIG. 150*a*.—A Typical Masonry Footing.

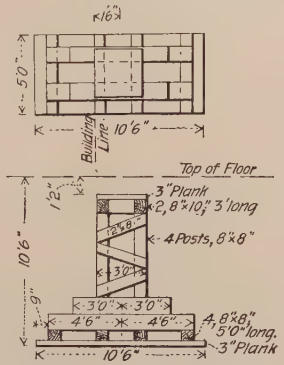


FIG. 150*b*.—Spread Footing of Timber Column.

preservation. The high price of timber, together with its relatively low transverse strength and the uncertainty of the future ground-water level, makes timber an undesirable material for use in permanent foundations. For temporary structures, such as exposition buildings, it is still used to some extent. Fig. 150*b* and Fig. 150*c* show the details of such a grillage when used under columns.

Another type, which was employed in some of the early

heavy Chicago buildings, consisted of a thick concrete platform continuous over the whole area of the building site, forming a

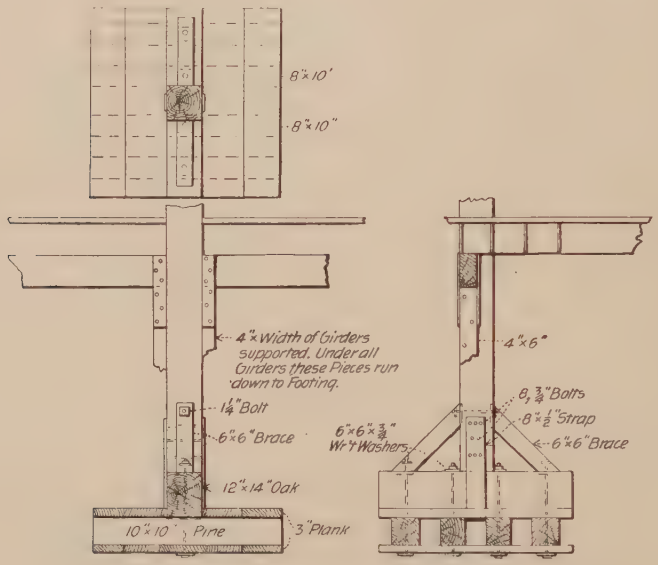


FIG. 150c.—Braced Column Footing.

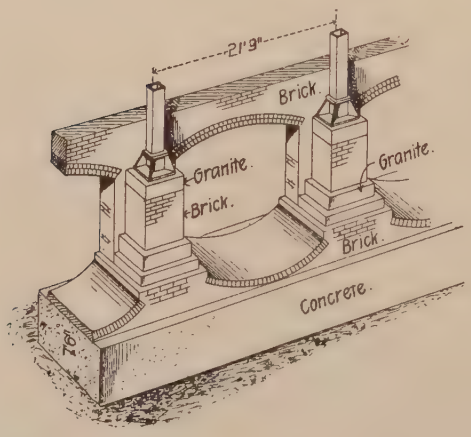


FIG. 150d.—Footing of Inverted Masonry Arch, Drexel Building, Philadelphia, Pa.

deep monolithic slab at the cellar-floor level and on which the columns and walls rested. The effect of variation in the magni-

tude of the concentrated loads was to crack the concrete bed into a number of independent footings and this was naturally followed by great irregularity in the settlement of various parts of the building. Hence this form of footing was never entirely satisfactory.

Another early type of spread foundation was that of the inverted masonry arch which was first used in the Drexel Building, Philadelphia, Pa., built in 1893, the details of which are shown in Fig. 150*d*. Another notable example of the use of this type was in the World Building, New York City. Both of these structures were among the early examples of the modern steel office building. In the Drexel Building the arches were made of brick, which distributed the column loads through continuous lines of concrete bases in the column rows, on the soil below. Although the brick masonry arch is no longer used, the principle is still employed in the reinforced-concrete arch footing described in Art. 159.

Art. 151. Modern Types of Spread Foundations

The two modern types of spread foundations are the steel I-beam grillage and the reinforced-concrete spread footing. The steel I-beam grillage dates back to the types described in the preceding article; but since it is still a standard type it is here described. The conditions surrounding its development are as follows: In the business district of Chicago the soil conditions are peculiar, made ground extending to a depth of about 14 feet below street grade while below that occurs a stratum of hard stiff clay 6 to 12 feet thick. Below this the clay, while having the same general characteristics as that above, becomes softer and remains so to a depth of 75 feet or more. The upper stiff clay makes a first-class foundation bed, but the softer clay below offers little supporting power.

After the great Chicago fire most of the new buildings were founded on masonry footings which rested on this hard clay stratum. Owing to the rapid increase in the size and weight of buildings it became necessary to increase the area of the base

of footings and this in turn compelled the use of deeper footings. As the bearing power of the soft clay below the hard stratum was small the only practicable method of obtaining this greater depth was to extend the footing up into the cellar; and thus the cellars soon became filled with pyramids of masonry, robbing them of valuable space. This, together with the fact that the masonry footings, on account of their large mass, formed too large a proportion of the total load and were expensive, started the search for a better type of footing for heavy loads. The type thus developed consisted of crossed layers of old steel rails, which were soon superseded by steel I-beams, both shapes being thoroughly embedded in concrete as a protection against rust.

Probably the first building in America to be built on a steel grillage foundation was the Montauk Block, Chicago, built in 1878, and designed by BURNHAM and ROOT, architects. The ordinary masonry footing was used for a part of the building, but to obtain space for the boiler a grillage of steel rails embedded in concrete was used in one part of the cellar. Soon after this date the price of steel I-beams dropped sufficiently to make them available for this purpose. On account of their larger section modulus for any given weight per foot, they are much more economical than rails, and in a short time I-beams were adopted exclusively. For very heavy loads built-up girders are often used in place of I-beams.

ART. 152. CONSTRUCTION OF I-BEAM GRILLAGES

In the construction of I-beam grillages two or more tiers are used, the exact number depending on the desired spread of base. Each tier is placed at right angles to the one below it and the load is carried to the soil through beam action. The individual beams of each tier should be held in place by cast-iron or gas-pipe separators, preferably the former. These separators should be placed near each end of the beams and at intermediate positions not over 5 feet apart. The beams should be spaced so as to give a clearance of not less than 3 inches, in order that the

concrete may readily be filled in between the beams; and not more than one and one-half times the width of the flange, in order to reduce the stresses in the concrete filling. The latter requirement cannot always be met.

Concrete should be filled in between the beams and also placed around the sides, top, and bottom of the grillage. The thickness of the bottom layer should not be less than 12 inches, and the top and sides should have a protective coating of at least 4 inches net thickness. If portland cement concrete is used the mixture should not be leaner than 1–3–6. A layer of cement grout of from $\frac{1}{2}$ to 1 inch in thickness should be placed between the tiers of beams.

Art. 153. Design of I-Beam Grillages

In designing a grillage the area of the column base and the column load will be known in advance. The grillage will be designed for the same load that is used for the base of the column, namely, the total dead load plus a certain percentage of the live load, the exact percentage depending on the kind of building and the number of stories. The weight of the grillage itself may usually be neglected.

As stated in Art. 149, a moderate amount of settlement is always to be expected, but care should be exercised to make this settlement uniform. To accomplish this, for any particular case, the unit-pressure on the soil should be the same for all footings. This is sometimes difficult to obtain on account of the very considerable difference in the proportion of live to dead load for different columns.

Engineers agree that it is essentially the dead load that causes the settlement: First, because it always acts with its maximum intensity; and second, because it is the first loading that comes on the foundation. Unequal settlements during erection due to dead load, are also troublesome in the case of steel buildings on account of the difficulties involved in fitting together the various members of the superstructure. For the above reasons most engineers design footings for equal unit-pressures under dead load, or under dead plus partial live loads.

In Table 153*a* the results are tabulated for the design of footings for an actual structure by using four formulas that are employed in current practice. Let D denote the dead load on the base of any column footing; T, total load on same footing; H, the dead load plus one-half the probable live load on the same footing; D', the dead load on the base of the critical column footing; T', the total load on the base of the critical column footing; H', the dead load plus one-half the probable live load on the base of the critical column footing; B, the safe unit bearing value of the soil; and A, the area of bearing of base of column footing for that column in which D denotes the dead load. The formulas are as follows:

McCullough,	$A = DT'/(BD')$
Schneider,	$A = DT'/(BD')$
Moran,	$A = HT'/(BH')$
Live + Dead	$A = T/B$

The McCullough formula gives uniform unit-pressure under dead load, and of a value which makes the pressure under that footing having the minimum ratio of live to dead load equal to the safe bearing value of the soil. Thus the critical column footing in this formula is the one having the minimum ratio of live to dead load.

The Schneider formula gives uniform unit-pressure under dead load, and of a value which makes the pressure under that footing having the maximum ratio of live to dead load equal to the safe bearing value of the soil. Thus the critical column footing in this formula is the one having the maximum ratio of live to dead load.

The Moran formula differs from Schneider's only in that equal unit-pressures occur under dead plus one-half probable live load.[1] The other formula is self evident.

[1] DANIEL E. MORAN explains the meaning of probable live load as follows: "The maximum probable load is the load which in the opinion of the designer will actually come upon the footings, and is to be determined by a study of the conditions which will obtain when the building is occupied. For instance, in a schoolhouse the number of children in each class room and the weight of desks, chairs, etc., may be determined with considerable accuracy and these loads will make

TABLE—153*a*.

		McCullough	Schneider	Moran	Live plus dead load	
Column 44	Area	67.6	103.6	81.2	67.6	Total load =473 000 lbs.
	Dead plus live load	7 000	4 550	5 830	7 000	Dead load =435 000 lbs.
	Dead plus ½ live load	6 720	4 380	5 590	6 720	Live load =38 000 lbs.
	Dead load	6 440	4 200	5 360	6 440	Dead + ½ live load =454 000 lbs.
						Dead load =92 percent
						Live load = 8 percent
Column 1	Area	58.3	89.3	87.6	86.4	Total load =605 000 lbs
	Dead plus live load	10 380	6 770	6 910	7 000	Dead load =375 000 lbs.
	Dead plus ½ live load	8 410	5 480	5 590	5 670	Live load =230 000 lbs.
	Dead load	6 440	4 200	4 280	4 340	Dead + ½ live load =490 000 lbs.
						Dead load =62 percent
						Live load =38 percent
Column 29	Area	66.6	102.2	90.8	84.0	Total load =588 000 lbs.
	Dead plus live load	8 820	5 750	6 480	7 000	Dead load =429 000 lbs.
	Dead plus ½ live load	7 620	4 970	5 590	6 050	Live load =159 000 lbs.
	Dead load	6 440	4 200	4 720	5 110	Dead + ½ live load =508 000 lbs.
						Dead load =73 percent
						Live load =27 percent
Column 24	Area	58.9	90.3	90.3	90.3	Total load =632 000 lbs.
	Dead plus live load	10 740	7 000	7 000	7 000	Dead load =379 000 lbs.
	Dead plus ½ live load	8 580	5 590	5 590	5 590	Live load =253 000 lbs.
	Dead load	6 440	4 200	4 200	4 200	Dead + ½ live load =505 000 lbs.
						Dead load =60 percent
						Live load =40 percent

Note: In the last vertical column are given the loads. The four horizontal lines for each footing give respectively the area of base in square feet and the pressure on the earth in pounds per square foot due to dead, live, and dead plus live load. The allowable unit-bearing on the ground is taken at 7000 pounds per square foot. In this example the probable live load is taken equal to the full live load.

The weak element of the McCullough formula is that although it gives under dead load the same unit-pressure for all foundations, yet for column No. 24 it gives a pressure of 8580 pounds per square foot for dead plus one-half live load and 10 740 pounds per square foot for dead plus live load, both of which are dangerously high when compared to a safe value of 7000 pounds per square foot. The Schneider formula, which also gives under dead load the same unit-pressure for all foundations, and a maximum dead plus live-load pressure of 7000 pounds per square foot, is very conservative. Neither of these two formulas gives any consideration to the live load in causing settlement. MORAN'S formula seems better in this respect since it recognizes the influence of the probable live load and gives to it one-half the weight that is given the dead load. In it the unit-pressure for dead plus one-half probable live load is so chosen that the maximum pressure under dead plus live load equals the safe bearing power of the foundation bed. The dead-plus-live-load formula gives entirely too much weight to live load, as is seen from the large variation in the dead-load stresses. For a further discussion on this subject see Engineering News, vol. 69, page 463, March 6, 1913, and page 687, April 3, 1913.

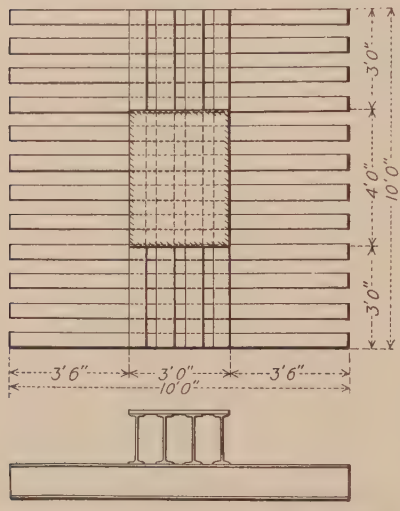


FIG. 153*a*.—Steel I-Beam Grillage for a Single Column.

the maximum probable live load. As a further illustration, in many schoolhouses there is an assembly room which is only used when the class rooms are vacant and consequently if class-room loads are used assembly-room loads should be omitted or *vice versa;* the greater one of these loadings to be used for the probable load." A full explanation of his method may also be found in the revised edition of KIDDER'S Architects' and Builders' Pocket Book.

In designing steel grillage foundations the following assumptions are made: First, the pressure from the footing is uniformly distributed over the bed; second, the pressure of one tier of beams on another is uniformly distributed over the latter; third, each tier acts independently of all other tiers; and fourth, the concrete filling and covering carries no stress, acting merely as a protection against corrosion.

For the single-column grillage the square base is the most economical shape. Where the possible width is restricted, as in the case of wall-column footings, the grillage should be made as nearly square as possible. Economy also results in using a minimum number of tiers.

EXAMPLE OF DESIGN OF SINGLE-COLUMN FOOTING.—Load = 600 000 lbs. Allowable pressure on foundation bed = 6000 lbs. per sq. ft. Size of column base = 3 × 4 ft. Required area of base = 600 000/6000 = 100 sq. ft. A base 10 ft. square is adopted. Assume two tiers of beams. For the top tier, the maximum bending moment $M = (600\,000/4)\ (5-2)12 = 5\,400\,000$ lb.-in. Using 16 000 lbs. per sq. in. as the safe unit-stress in the outer fiber, the total section modulus required $= I/e = 5\,400\,000/16\,000 = 337$ in^3. Trying various combinations of beams, the following results are obtained:

No.	Number of beams	I/e required	Size of beam	I/e furnished	Width of flange	Clearance
1	3	112.3	20″–65 lb.	117.0	6.25 in.	8.6 in.
2	4	84.2	18″–55 lb.	88.4	6.00 in.	4.0 in.
3	5	67.4	15″–55 lb.	68.1	5.75 in.	1.8 in.

The choice lies between Nos. 1 and 2, since No. 3 does not give sufficient clearance. The weight favors No. 1, being 250 pounds lighter, while No. 2 gives a more satisfactory clearance and has less depth, thus saving on concrete filling and also on excavation work.

Cost of 250 lbs. of steel at 2½ cents	$6.25
Cost of a 2-in. thickness of concrete	$1.65
Amount saved by using design No. 1	$4.60

For the lower tier: Max. $M = (600\,000/4)\ (5-1.5)12 = 6\,300\,000$ lb.-in. Total required $I/e = 6\,300\,000/16\,000 = 394$ in^3. The following results are obtained by trying various combinations of beams:

No.	Number of beams	I/e required	Size of beam	I/e furnished	Width of flange	Clearance
1	10	39.4	12″–40 lb.	41.0	5.21 in.	7.5 in.
2	12	32.9	12″–31½ lb.	36.0	5.00 in.	5.5 in.
3	14	28.2	12″–31½ lb.	36.0	5.00 in.	3.8 in.
4	16	24.6	10″–25 lb.	24.4	4.66 in.	3.0 in.

The choice lies between Nos. 2 and 4; the latter has 220 pounds more steel but the clearance is better and a 2-inch depth of concrete is saved.

Cost of 220 lbs. of steel at 2½ cents..... $5.50
Cost of a 2-inch thickness of concrete..... $4.50

Amount saved by using design No. 2..... $1.00

ART. 154. DESIGN OF DOUBLE-COLUMN FOOTINGS

Where the two-column loads are equal the base of the footing should be rectangular in shape and symmetrical about a line

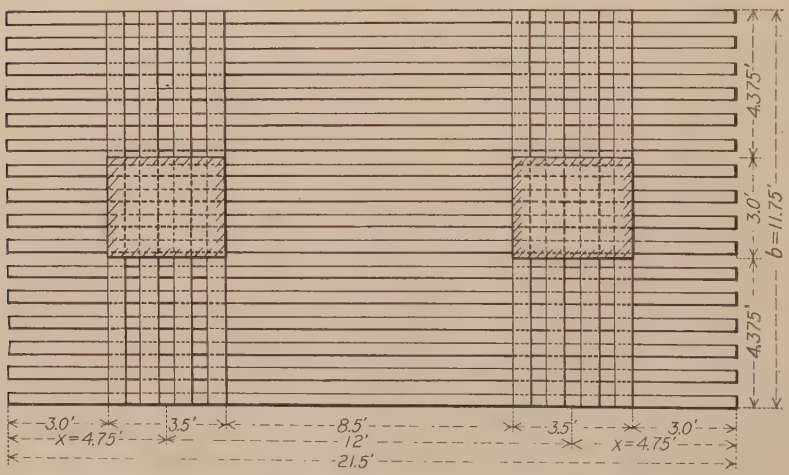


FIG. 154a.—Double-Column Footing of Steel I-Beams.

midway between the columns. The total area of the base having been determined and the distance between columns fixed, the proportion of length to breath for the base of footing should be such that the moment in the lower tier of beams under the column centers equals that at a point midway between the col-

umns. This makes the three maximum moments approximately equal, and gives the greatest economy of material.

EXAMPLE OF DESIGN OF DOUBLE-COLUMN FOOTING, EQUAL LOADS.—Column loads = 500 000 lbs. Column spacing = 12 ft. Allowable pressure on ground = 4000 lbs. per sq. ft. Size of column bases = $3\frac{1}{2} \times 3$ ft. Allowable unit-stress in beams = 16 000 lbs. per sq. in. To get the value x that will make the three moments equal, $500\,000(6-x)/2 = 500\,000x^2/2(6+x) - 500\,000(\frac{7}{16})$, whence $x = 4.77$ ft. Required bearing area of base = 1 000 000/4000 = 250 sq. ft. Using a value of x of 4.75 ft., $b = 250/(12+2\times4.75) = 11.63$ ft.; say 11.75. Let two tiers of beams be assumed. Computing for top tier: Max. $M = 500\,000(11.75-3)12/8 = 6\,560\,000$ lb.-in. Total required $I/e = 6\,560\,000/16\,000 = 410$ in^3. After trying various combinations of beams, the results are:

No.	Number of beams	I/e required	Size of beam	I/e furnished	Width of flange	Clearance
1	3	136.7	24″–80 lb.	173.9	7.0 in.	10.5 in.
2	4	102.5	20″–65 lb.	117.0	6.25 in.	5.3 in.
3	5	82.0	18″–55 lb.	88.4	6.0 in.	3.0 in.

No. 2 will be adopted.

For lower tier the three positions of maximum bending moment are at the center and 4.45 ft. from each end. M at center = 500 000 (6 − 5.375)12 = 3 750 000 lb.-in. M at 4.45 ft. from the end =

$$\left[\frac{500\,000}{10.75} \cdot \frac{4.45^2}{2} - \frac{500\,000}{3.5} \cdot \frac{1.45^2}{2}\right] 12 = 3\,750\,000 \text{ lb.-in.}$$

Total required $I/e = 3\,750\,000/16\,000 = 234$. Upon trying various combinations of beams, the results are found to be:

No.	Number of beams	I/e required	Size of beam	I/e furnished	Width of flange	Clearance
1	12	19.5	9″–21 lb.	18.9	4.33 in.	8.1 in.
2	14	16.7	9″–21 lb.	18.9	4.33 in.	6.1 in.
3	16	14.6	8″–18 lb.	14.2	4.0 in.	5.1 in.
4	18	13.0	8″–18 lb.	14.2	4.0 in.	4.1 in.

No. 3 will be adopted.

When the column loads are not equal the center of gravity of the base of the grillage is usually made to coincide with the line of action of the resultant of the two column loads by making the base a trapezoid; or, if the loads are nearly equal, it may be done

by using a rectangular shape and making the cantilever end at the heavy load longer than the other cantilever end. The trapezoidal shape may be obtained either by using a larger number of beams at the heavy load end, or by using the same number of beams and spacing them more closely at one end than at the other. A combination of the two methods is sometimes used.

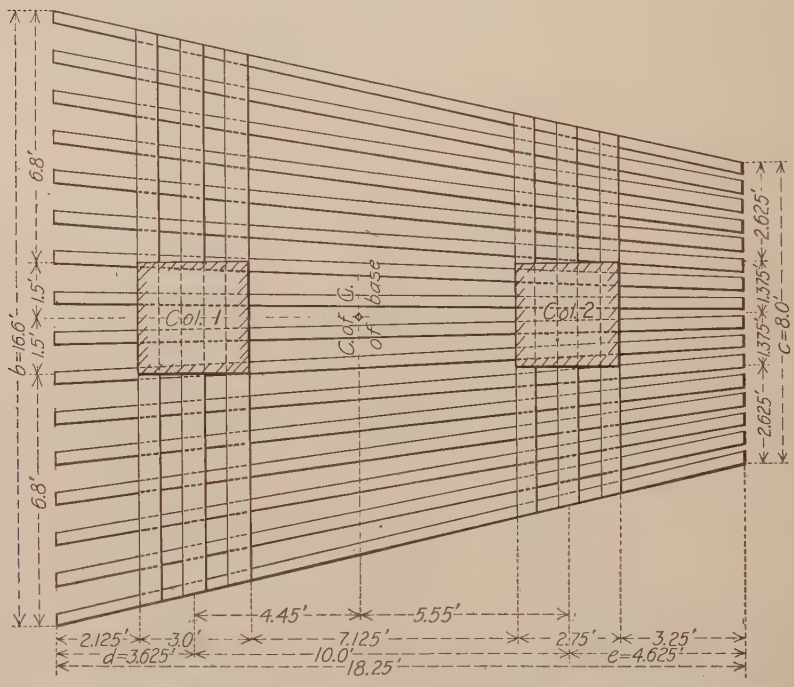


FIG. 154*b*.—Steel I-beam Grillage for Two Columns Supporting Unequal Loads. The load on Column No. 1 is 500 000 lbs; that on No. 2 is 400 000 lbs.

If the proportions of the base are so fixed that the bending moment under the center of each column equals that at the center of gravity of the base, the three maximum moments in the lower tier of grillage will be closely equal; this condition gives approximately the minimum amount of material. The most satisfactory method of determining the dimensions to secure this result is by trial.

EXAMPLE OF DESIGN OF DOUBLE-COLUMN FOOTING, UNEQUAL LOADS.—Column loads and spacing as shown in Fig. 154 *b*. Allowable pressure on foundation bed = 4000 lbs. per sq. ft. Size of column bases as shown in Fig. 154*b*. Allowable unit-stress in beams = 16 000 lbs. per sq. in. Required bearing area of base = 900 000/4000 = 225 sq. ft. Distance from Column No. 1 to resultant of both column loads = 400 000 × 10/900 000 = 4.45 ft.

After a few trials it was found that the moments under the centers of the two columns and under the center of gravity of base—line of action of resultant of two column loads—were approximately equal when $d = 3.625$ ft., and $e = 4.625$ ft.

To get b and c: $(b+c)18.25/2 = 225$ and $(18.25/3)\ (b+2c)/(b+c) = 3.625+4.45 = 8.075$. Solving these two equations simultaneously we find that approximately $b = 16.6$ ft. and $c = 8.0$ ft.

Using two tiers of beams, the computations for the upper tier under Column No. 1 give:

Max. $M = (500\,000/8)\ (14.88-3)12 = 8\ 910\,000$ lb.-in. Total required $I/e = 557$ in^3. After trying various combinations of beams, the results are as follows, and No. 1 is adopted:

No.	Number of beams	I/e required	Size of beam	I/e furnished	Width of flange	Clearance
1	3	186.0	24″–90 lb.	186.5	7.13 in.	7.3 in.
2	4	139.5	24″–80 lb.	173.9	7.0 in.	2.7 in.

For the upper tier under Column No. 2:

Max. $M = (400\,000/8)\ (10.18-2.75)12 = 4\,458\,000$ lb.-in. Total required $I/e = 279$ in^3. Trying various combinations of beams gives the following results, No. 1 being adopted:

No.	Number of beams	I/e required	Size of beams	I/e furnished	Width of flange	Clearance
1	3	93.0	18″–60 lb.	93.5	6.10 in.	7.3 in.
2	4	69.7	18″–55 lb.	88.4	6.00 in.	3.0 in.

In designing the lower tier and running all beams full length, let x = the distance from the left end of grillage to section in question, the expression for bending moments under Column No. 1, between the two columns, and under Column No. 2, are respectively as follows:

$$M(\text{col. No. 1}) = \frac{4000}{2} \cdot \frac{x^2}{3} \cdot (49.8 - 0.471x) - \frac{500\,000}{3} \cdot \frac{(x-2.125)^2}{2}$$

$$M \text{ (between cols.)} = \frac{4000}{2} \cdot \frac{x^2}{3} \cdot (49.8 - 0.471x) - 500\,000(x-3.625)$$

$$M(\text{col. No. 2}) = \frac{4000}{2} \cdot \frac{x^2}{3} \cdot (49.8 - 0.471x) - 500\,000(x-3.625) - \frac{400\,000}{2.75} \cdot \frac{(x-12.25)^2}{2}$$

To get the values of x for the maximum value of M in each of the above equations equate dM/dx equal to zero, which gives 3.42, 8.58 and 13.91 ft., respectively. Substituting these three values of x in the preceding equations, the corresponding values of M are, 236 000 lb.-in., 231 000 lb.-in., and 232 000 lb.-in. The maximum maximorum is therefore 236 000 lb.-in. Total required $I/e = 177$ in.[3] Trying various combinations of beams, there is obtained:

No.	Number of beams	I/e required	Size of beam	I/e furnished	Width of flange	Clearance
1	10	17.7	9″–21 lb.	18.9	4.33 in.	5.9 to 17.3 in.
2	12	14.7	8″–20¼ lb.	15.0	4.08 in.	4.3 to 13.6 in.
3	14	12.7	8″–18 lb.	14.2	4.00 in.	3.1 to 10.9 in.
4	16	11.0	7″–17½ lb.	11.2	3.66 in.	2.5 to 9.3 in.

No. 4 will be adopted.

Reinforcing bars should be placed in the concrete near the upper surface for the wider half of the footing.

ART. 155. DISTRIBUTION OF PRESSURE ON BASE

There is some question regarding the error involved in the assumption that the pressure from the footing is uniformly distributed on the ground. Taking the case of the single-column square footing it is evident that the base of the footing will assume a saucer-like shape, and as a consequence the pressure will be a maximum at the center and a minimum at the outside. The law governing the variation of pressure will

depend on the relative deflections of different points on the base of the footing, as well as on the modulus of compressibility of the soil and the thickness of the compressible stratum. Where the modulus is low and the thickness considerable, the slight difference in total deformation at different points will cause but a slight difference in pressure. Where the soil is compressible but inelastic, or soft and subject to lateral flow, a fairly uniform distribution of pressure quickly obtains.

Where the material has a high modulus of compressibility, as in shale or rock, the footing should be designed for stiffness as well as for strength or else the surface of the material should be shaped to fit the curve taken by the base of the footing when fully loaded, otherwise the pressure will be very unevenly distributed. For example, by using a stress-strain diagram of the values obtained in the foundation tests of the St. Paul Building, New York City (see Engineering Record, vol. 33, page 388, May 2, 1896), a theoretical solution shows that for the typical steel-grillage footing the pressure varies from a maximum at the center to approximately zero at the outside. The material on which the above foundation tests were made consisted of very compact sand, while the whole area of the lot was covered with a layer of concrete and steel beams buried in concrete, the tests being made through a hole.

Art. 156. Steel Grillage Foundations

Most of the grillages used in the foundations for the Phelan Building, San Francisco, were 15 feet square, and made with two cross tiers of I-beams from 18 to 24 inches in depth, or with an upper tier of built-up girders and a lower tier of I-beams, as shown in Fig. 156*a*. The complete grillage plan is shown in Fig. 156*b*.

[1] "All footings are made with a bed of concrete 12 inches thick and 12 inches wider and longer than the dimensions of the first tier of grillage beams. In the upper part of the concrete there are two full-length rectangular grooves transverse

[1] Engineering Record, vol. 57, page 366, March 28, 1908.

to the lower tier of grillage beams. In each groove a $3 \times 3 \times \frac{5}{16}$-inch angle was carefully leveled with the upper edge of its vertical flange truly horizontal and $\frac{3}{4}$ inch above the surface

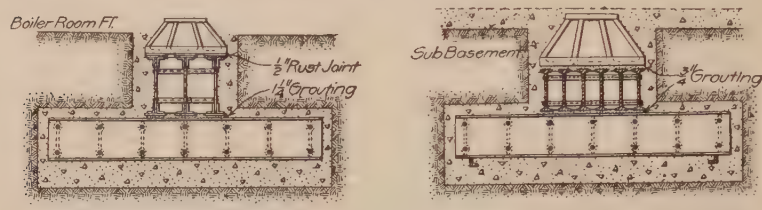


FIG. 156*a*.—Footings with Plate Girders and I-beams in Double Tiers.

of the concrete. These serve as leveling bars to receive the lower flanges of the grillage beams and insure their exact height. The spaces between the beams and the concrete footings were grouted, the second tier of beams was shimmed $\frac{3}{4}$ inch above the top flanges of the lower tier and grouted, the cast-iron pedestals were set $\frac{3}{4}$ inch above the top flanges of the distributing beams and grouted, and a solid mass of concrete was

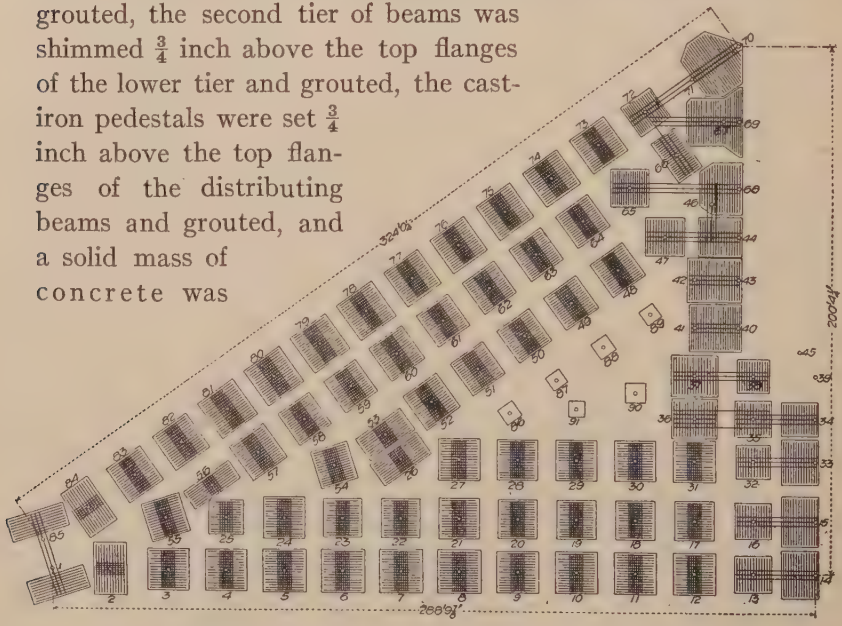


FIG. 156*b*.—Grillage Plan of Phelan Building, San Francisco, Cal.

filled in 6 inches around the outer edges of the beams and pedestals and up to the cellar floor, completely enclosing and protecting all the substructure steel work."

Fig. 156*c* illustrates a very heavy grillage foundation for four columns of the Curtis Building, Philadelphia. It was necessary to use a single grillage for the four columns because of the short distances between the latter. The distributing girders for Columns Nos. 254 and 255 have $48\times\frac{1}{16}$-inch webs

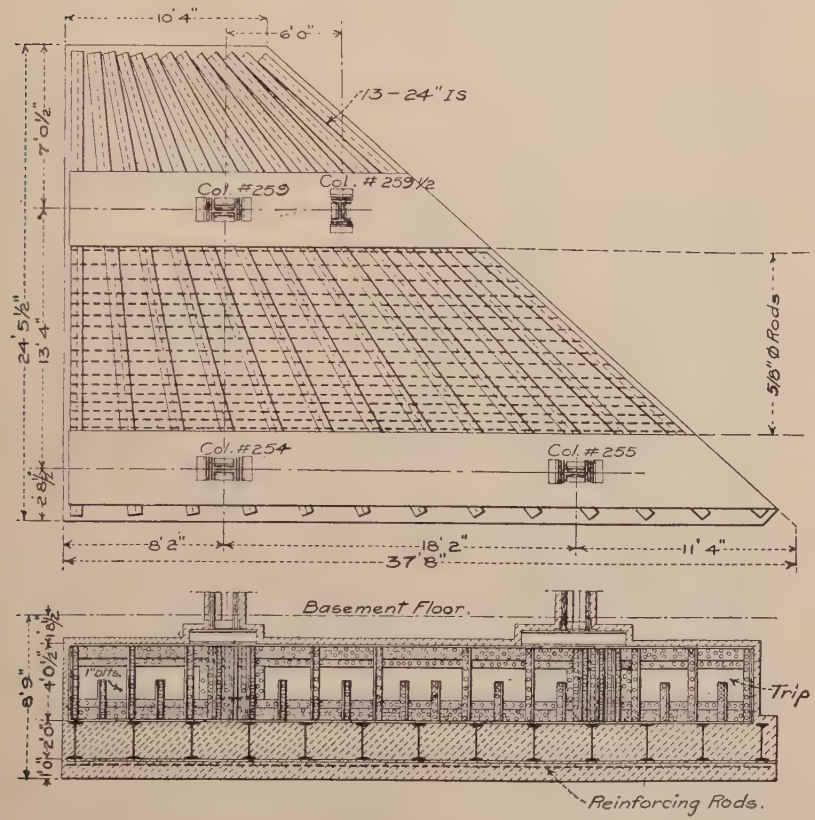


FIG. 156*c*.—Special Footing for Four Columns, Curtis Building, Philadelphia.

reinforced by $5\times3\times\frac{3}{8}$-inch vertical stiffener angles and two $13\times\frac{1}{2}$-inch vertical side plates, and the top flanges of the girders are connected by transverse tie plates. The column loads are transmitted to the triple distributing girders by bolsters made of solid slabs of plain square steel billets

which are bolted to the upper flanges of the girders. The concrete footing is reinforced with rods for part of the base, due to the fact that the I-beams are there a considerable distance apart, thus developing beam action in the concrete.

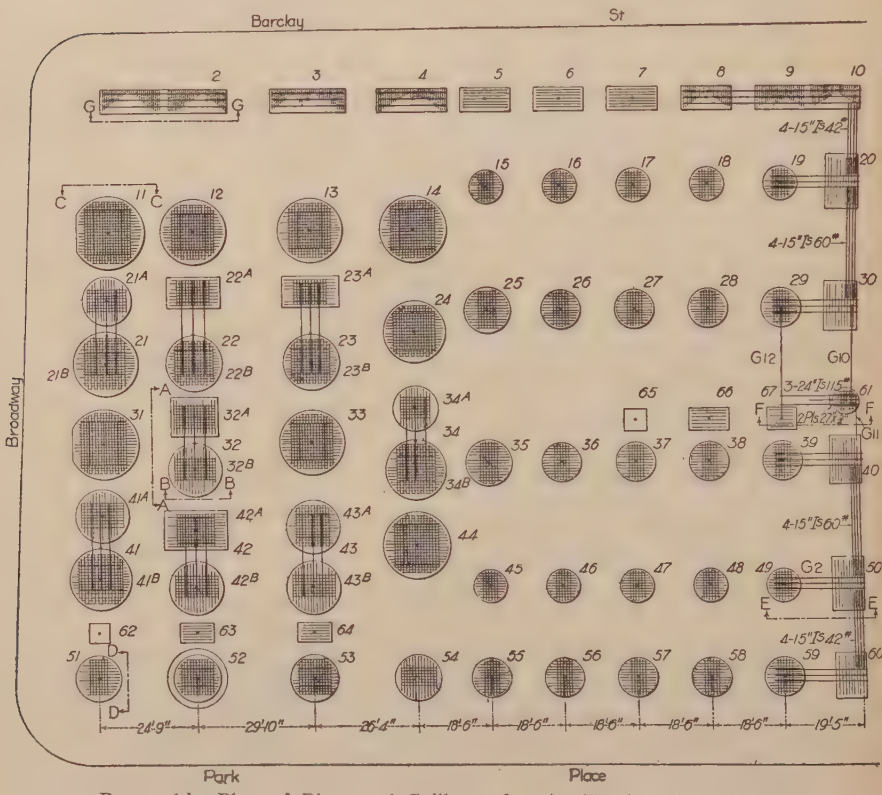


FIG. 156*d*.—Plan of Piers and Grillages for the Woolworth Building.

The Woolworth Building, New York City, is founded on solid rock 115 feet below the curb level. The loads are carried from the columns to bedrock through grillage footings resting on reinforced-concrete piers. Fig. 156*d* shows the general lay out for the foundation, while 156*e* shows some of the details.

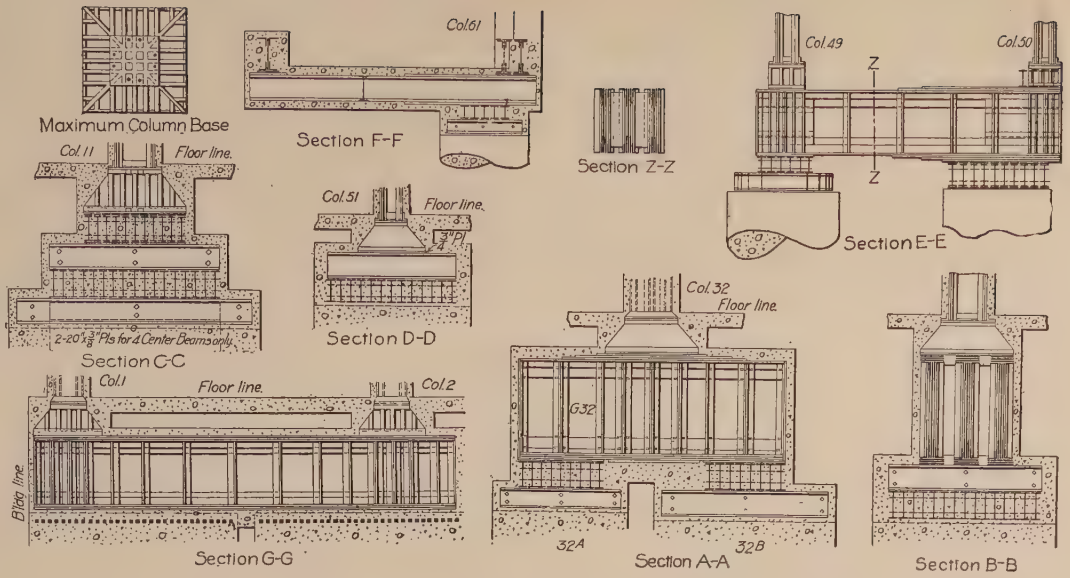


FIG. 156*e*.—Column Footings of Plate Girders and Grillages, Woolworth Building, New York City

For Column No. 11 the diameter of the pier is 18 feet 9 inches and the column load is 9,423,000 pounds. For this column the grillage is made of four tiers of 24 inch I-beams with flange reinforcement plates for the top tier and web reinforcement plates for the bottom tier. Section *E-E* shows the common method of supporting wall columns when the foundation must be kept back of the property line.

ART. 157. DESIGN OF REINFORCED-CONCRETE SPREAD FOUNDATIONS

Instead of serving merely as a protection for the steel, concrete may be made to take a part of the load by using a reinforced-concrete footing in place of the I-beam grillage, thus lessening the cost of the foundation. Another advantage possessed by a reinforced-concrete foundation is that it can be cast in any shape or form desired. It may be in the form of a flat slab or of the slab-and-beam type (Fig. 159*a*). The former uses more concrete, while in the latter the form work is more expensive. For some interesting modifications of the elementary type the reader is referred to Art. 159.

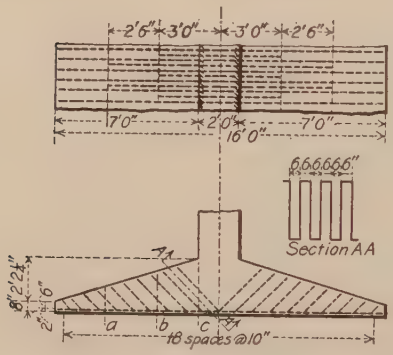


FIG. 157*a*.—Reinforced-concrete Wall Footing.

DESIGN OF A REINFORCED-CONCRETE WALL FOOTING.—Assuming the load to be 64 000 pounds per linear foot of wall and the allowable bearing on the soil 4000 pounds per square foot, the width of footing will be $64\,000/4000 = 16$ feet. The thickness of the wall is 2 feet (Fig. 157*a*). The footing will be designed at three sections, at *a*, $5\frac{1}{2}$ feet from the center of the wall, at *b*, 3 feet from the center, and at *c*, 1 foot from the center. Taking a 1-foot length of footing the vertical shears and bending moments will be as follows:

$$V_a = 4000 \times 2\tfrac{1}{2} = 10\,000 \text{ lb.} \qquad M_a = \frac{4000 \times 2.5^2 \times 12}{2} = 150\,000 \text{ lb.-in.}$$

$$V_b = 4000 \times 5 = 20\,000 \text{ lb.} \qquad M_b = \frac{4000 \times 5^2 \times 12}{2} = 600\,000 \text{ lb.-in.}$$

$$V_c = 4000 \times 7 = 28\,000 \text{ lb.} \qquad M_c = \frac{4000 \times 7^2 \times 12}{2} = 1\,176\,000 \text{ lb.-in.}$$

A 1–2–4 concrete will be used, with an allowable compressive unit-stress[1] in the concrete of $f_c = 600$ lbs. per sq. in. and an allowable tensile unit-stress in the steel of $f_s = 16\,000$ lbs. per sq. in. The ratio of the modulus of elasticity of steel to that of concrete will be assumed as $n = 15$. The depth to center of steel rods necessary to give a compressive stress in the concrete of 600 lbs. per sq. in. is given by the formula, $d = \sqrt{M/(Rb)}$, in which $R = f_c kj/2$. In the latter formula[2] $k = \sqrt{2pn + (pn)^2} - pn$ and $j = 1 - k/3$; $p = \frac{1}{2}/\frac{f_s}{f_c}\left(\frac{f_s}{nf_c} + 1\right)$. The work involved in getting the value of R will be greatly reduced by using the diagrams found in TURNEAURE and MAURER'S Reinforced-Concrete Construction. For the problem at hand the value of R is 95. Solving for d, $d_a = \sqrt{150\,000/(95 \times 12)} = 11.5$ in.; $d_b = \sqrt{600\,000/(95 \times 12)} = 23.0$ in.; and $d_c = \sqrt{1\,176\,000/95 \times 12)} = 32.1$ in. ($32\frac{1}{4}$ in. being adopted). As it is inadvisable to use a depth at any section less than about 6 inches the form shown in Fig. 157 *a* will be adopted. The steel in the bottom will be given a 2-in. insulation.

The area of steel required at each section is given by the formula, $A = M/(f_s jd)$. Using the values of d obtained above, so that the footing be equally strong in tension and compression:

$$A_a = 150\,000/(16\,000 \times 0.88 \times 11.5) = 0.92 \text{ square inch},$$
$$A_b = 600\,000/(16\,000 \times 0.88 \times 23\ ) = 1.85 \text{ square inches},$$
$$A_c = 1\,176\,000/(16\,000 \times 0.88 \times 32.1) = 2.60 \text{ square inches}.$$

Using a rod spacing of 3 inches center to center there will be 4 rods in one foot of length of the footing. The required area of each rod will be $2.60/4 = 0.650$ sq. in. A $\frac{3}{4}$-inch square

[1] In a wedge-shaped beam the greater principal stress at the outer fibers acts parallel to the upper surface of the beam and with an intensity equal to the maximum normal stress on a vertical plane divided by $\cos^2\alpha$, in which α is the angle of inclination of the upper surface of the beam; hence, the allowable bending unit stress should be taken equal to the safe compressive stress in the concrete multiplied by $\cos^2\alpha$.

[2] Based upon the assumption that the normal stress in the concrete on any vertical section varies as a straight line and that the stress in the steel equals n times the stress in the concrete. For formulas based on a different assumption see Proc. Am. Soc. C. E., vol. 39, page 2067, Nov. 1913.

twisted rod, giving an area of 0.660 sq. in. will be adopted. Three rods will furnish the required area at b, while two rods will furnish that required at a; hence certain of the rods may be bent up or cut off as shown in Fig. 157a.

Using an allowable bond unit-stress of 140 lbs. per sq. in. of rod surface the necessary length of rod to develop full strength is $(16\,000\times 0.66)/(140\times 3.25) = 23.2$ in. Computing the bond stress in the rods by the formula [1] $u = (Sd - M \tan \alpha)/(jd^2\ \Sigma o)$ in which $\tan \alpha$ is the slope of the upper surface of the footing and Σo the perimeter of the rods at the section in question, the values are as follows:

$u_a = (10\,000\times 15.4 - 150\,000\times 0.312)/(0.88\times \overline{15.4^2}\times 6.50) = 79$ lb./sq. in.

$u_b = (20\,000\times 24.75 - 600\,000\times 0.312)/(0.88\times 24.75^2\times 9.75) = 59$ lb./sq. in.

$u_c = (28\,000\times 32.25 - 1\,176\,000\times 0.312)/(0.88\times \overline{32.25^2}\times 13) = 45$ lb./sq. in.

All of these values are well below the safe limit of 140 lbs. per sq. in.

Assuming that the concrete takes no longitudinal tension the maximum intensity of diagonal tension is given by the formula $t = (Sd - M \tan \alpha)/(jd^2)$. A shorter method of computing the maximum diagonal tension is by taking the bond stress values and multiplying them by the perimeter of the rods. Thus,

$$t_a = (79\times 6.50)/12 = 43 \text{ lbs. per sq. in.}$$

$$t_b = (59\times 9.75)/12 = 48 \text{ lbs. per sq. in.}$$

$$t_c = (45\times 13.0)/12 = 49 \text{ lbs. per sq. in.}$$

Although conservative specifications limit the allowable diagonal tension to 40 pounds per square inch, the above can be safely carried by the concrete without reinforcement, but to illustrate the method stirrups will be designed to carry all of this tension. Placing the stirrups on a 45-degree slope and using $\frac{3}{8}$-inch square twisted rods with two prongs in a 12-inch length, as shown in Fig. 157a, the strength of one line of stirrups in a 12-inch length will be $16\,000\times(\frac{3}{8})^2\times 2 = 45\,000$ pounds. Denoting the horizontal distance between rows of

[1] Only approximately true when p is not constant.

stirrups by s the formula is, $s = 4500/(12 \times t \times \cos 45°)$, giving

$$S_a = 4500/12 \times 43 \times 0.707 = 12 \text{ inches.}$$
$$S_b = 4500/12 \times 48 \times 0.707 = 11 \text{ inches.}$$
$$S_c = 4500/12 \times 49 \times 0.707 = 10 \text{ inches.}$$

A uniform spacing of 10 inches will be adopted.

In this type of beam the maximum intensity of vertical shear occurs at the top and equals $f_c \tan \alpha$, where α is the inclination of the upper surface of the slab. The shearing stress is therefore $600 \times 0.312 = 187$ pounds per square inch.

ART. 158. DESIGN OF REINFORCED-CONCRETE COLUMN FOOTINGS

The stresses in a reinforced-concrete footing for a column are due more to flat-slab action than to beam action and hence are much less determinate than in the wall footing. However, the stresses may be approximately analyzed by either flat-slab or beam formulas. The former method is not entirely satisfactory, due partly to the necessary approximations of any formulas based on the theory of the flat plate, and partly to the tedious computations involved unless specially prepared tables or diagrams are used. For an example of the design of a footing based on the flat-slab principle see page 644 of the second edition of TAYLOR and THOMPSON'S Concrete, Plain and Reinforced.

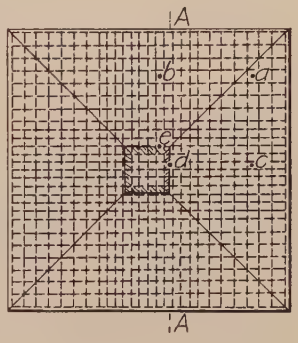


FIG. 158*a*.—Column Footing of Reinforced Concrete.

Where beam formulas are used it is generally assumed that the section of maximum bending moment and shear is at the outer face of the column. If the footing has a two-way reinforcement the stress cannot be uniformly distributed over this section. For instance, looking at Fig. 158 *a*, the load from the soil at point *c* will evidently go to the column through *dc*

acting as a cantilever beam. On the other hand a part of the load at *a* will first go to some point as *c* through *ac* acting as a beam, and the balance to some point as *b* through *ab* acting as a beam. The part which goes to *c* will then go to *d* through *cd* acting as a beam, while the part which goes to *b* will go to *e* through *be* acting as a beam. Thus it is evident that the stress along the plane *A–A* will vary from a maximum at the column face to a minimum near the sides of the footing.

From experiments made in the testing laboratory at the University of Illinois, A. N. TALBOT summarizes the proper method of design as follows: [1] "For footings having projections of ordinary dimensions, the critical section for the bending moment for one direction (which in two-way reinforced concrete footings is to be resisted by one set of bars) may be taken to be at a vertical section passing through the face of the pier. In calculating this moment, all the upward load on the rectangle lying between a face of the pier and the edge of the footing is considered to act at a center of pressure located at a point halfway out from the pier, and half of the upward load on the two corner squares is considered to act at a center of pressure located at a point six-tenths of the width of the projection from the given section. . . .

"With two-way reinforcement evenly spaced over the footing, it seems that the tensile stress is approximately the same in bars lying within a space somewhat greater than the width of the pier and that there is also considerable stress in the bars which lie near the edges of the footing. For intermediate bars stresses intermediate in amount will be developed. For footings having two-way reinforcement spaced uniformly over the footing, the method proposed, for determining the maximum tensile stress in the reinforcing bars, is to use in the calculation of resisting moment at a section at the face of the pier the area of all the bars which lie within a width of footing equal to the width of pier plus twice the thickness of footing, plus half the remaining distance on each side to the edge of the footing. This method gives results in keeping with the results

[1] Bulletin No. 67, Engineering Experiment Station, University of Illinois.

of tests. When the spacing through the middle of the width of the footing is closer, or even when the bars are concentrated in the middle portion, the same method may be applied without serious error. Enough reinforcement should be placed in the outer portion to prevent the concentration of tension cracks in the concrete and to provide for other distribution stresses.

"The method proposed for calculating maximum bond stress in column footings having two-way reinforcement evenly spaced, or spaced as noted in the preceding paragraph, is to use the ordinary bond-stress formula, and to consider the circumferences of all the bars which were used in the calculation of tensile stress, and to take for the external shear that amount of upward pressure or load which was used in the calculation of the bending moment at the given section." In the preceding discussion the slab is assumed to have a horizontal upper surface.

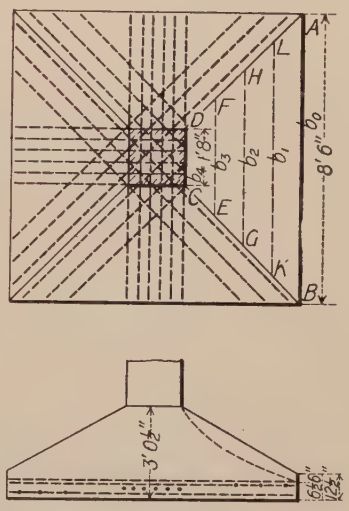


FIG. 158*b*.—Reinforcement for Column Footing.

DESIGN OF A FOUR-WAY REINFORCED FOOTING.—A footing with four-way reinforcement (Fig. 158*b*) is more susceptible of a rational analysis than the two-way reinforced footing. Tests by A. N. TALBOT (see previous reference) show that this type gives a somewhat stronger footing than the two-way type.

Assuming the load to be 210 000 pounds and the allowable bearing on the soil 3000 lbs. per sq. ft., the area of the footing will be 210 000/3000 = 70 sq. ft. A base8 feet 6 inches square will be used. The column base will be assumed as 20 inches square.

In this design the part *ABCD* in Fig. 158 *b* will be assumed to act as a free cantilever about *CD*, as will also *ABEF*, *ABGH*

and $ABKL$ about EF, GH and KL respectively; in other words, it will be assumed that there is no stress on the planes AD and BC. Dividing the horizontal distance between AB and DC into four equal parts by the lines b_1, b_2 and b_3, the lengths of the lines b_0, b_1, b_2, b_3 and b_4 are respectively 8.50, 6.79, 5.08, 3.37 and 1.67 ft. Let A_1, A_2, A_3 and A_4 represent respectively the areas of the base of the footing to the right of the b lines of the corresponding subscripts, then their values will be $A_1 = 6.54$, $A_2 = 11.6$, $A_3 = 15.2$ and $A_4 = 17.35$, all expressed in square feet.

The upward pressure from the soil is $210\,000/(8.5)^2 = 2910$ lbs. per sq. ft. The shears on the sections b_1, b_2, b_3 and b_4 are respectively 19 000, 33 000, 44 200 and 50 500 pounds. The moment of the upward pressure to the right of and about b_1 is $19\,000 \times \frac{2 \times 8.5 + 6.79}{8.5 + 6.79} \times \frac{0.854}{3} \times 12 = 101\,000$ lb.-in. The moments of the forces to the right of and about b_2, b_3 and b_4 are respectively 376 000, 775 000, and 1 267 000 lb.-in. Using an allowable unit stress for the rods of 16 000 lbs. per sq. in. and for the concrete of $650 \cos^2 \alpha = 500$ (approximately) lbs. per sq. in., in which α is the angle made by the upper surface with the horizontal, the values of d as given in the formula $d = M/(Rb)$ are $d_1 = 4.2$, $d_2 = 9.3$, $d_3 = 16.4$ and $d_4 = 29.8$ in.

Using the formula $A = M/(f_s jd)$ to get the required area of cross-section of steel at b_1, b_2, b_3 and b_4, the respective values are 1.69, 2.83, 3.30 and 2.97 sq. in. Assuming 12 square twisted rods, the required area of each one is $3.30/12 = 0.275$ sq. in. A $\frac{9}{16}$-inch rod furnishes an area of 0.316 sq. in. The rods will be placed as shown in Fig. 158*b*, each layer being $1\frac{1}{2}$ in. above the one below it.

The ordinates to the curved line in Fig. 158*b* represent the required depths, but, as shown in the same illustration, the depths adopted will be greater than these.

The bond stresses as given by the formula $u = (Sd - M \tan \alpha)/(jd^2 \Sigma o)$ are 48, 50, 44 and 36 lbs. per sq. in. for the sections b_1, b_2, b_3, and b_4 respectively.

The maximum unit shear is $f_c \tan \alpha = 500 \times 0.586 = 293$ lbs.

per sq. in. This is a rather high value but as it occurs at the point of maximum compression and so does not develop a heavy diagonal tension, it may be considered safe.

Assuming that the concrete takes no direct tension the maximum diagonal tension for each section, as given by the formula $t=(Sd-M \tan \alpha)/(bjd^2)$, is $t_1=16$, $t_2=22$, $t_3=29$ and $t_4=48$ lbs. per sq. in. Hence stirrups are required for only a short distance from the face of the column. The method of design of the same is treated in Art. 157 and will not be repeated here.

The design of the slab-and-beam type of footing follows closely the method of design of slabs and beams in building construction. The slab serves as a beam to carry the load from the soil to the beam, the span being taken as the distance center to center of beams; and the latter, acting as cantilevers, carry it to the column. Where the beams have constant cross-sections the formulas for stresses as derived in any standard treatise on reinforced concrete are applicable, and where tapered, the formulas given in Art. 157 may be used.

Where one footing serves for two columns, the method of obtaining the shape of footing, as well as the shears and bending moments, is similar to that for the I-beam grillage (Art. 154), while the standard formulas are applicable in finding the stresses. On page 647 of the second edition of TAYLOR and THOMPSON'S Concrete, Plain and Reinforced, an example of this type of footing is worked out.

ART. 159. CONCRETE SPREAD FOUNDATIONS

Two standard forms of the reinforced-concrete spread footings used for the column foundations of a railway terminal station at Atlanta, Ga., are shown in Fig. 159*a*. The one illustrated on the left was used for 20×24-inch columns and was in the form of a truncated pyramidal slab reinforced with bars and stirrups. The one shown on the right was of the beam-and-slab type. The details are sufficiently shown to require no explanation.

The 125-foot concrete block chimney for the St. Joseph's Home, Chicago, was founded on a blue clay, the base of the foundation extending about 5 feet below the surface of the ground. The footing, shown in Figs. 159*b* and *c*, consists of a circular slab 24 feet in diameter and 10 inches thick, on which is built a box with a square outer surface 8 feet 3 inches on a side, and with an octagonal inner surface about 6 feet 7

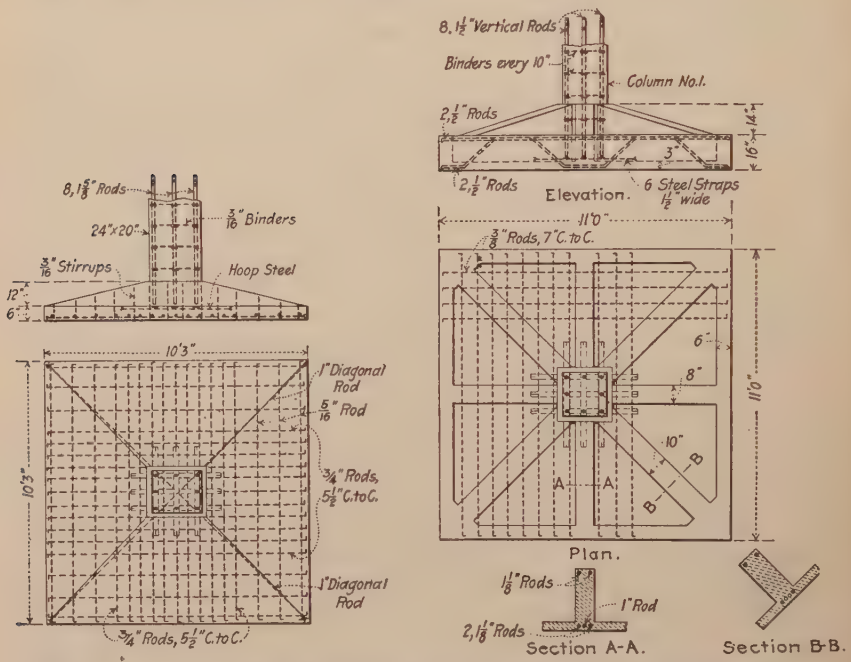


FIG. 159*a*.—Pyramidal and Ribbed Slab Footings of Reinforced Concrete, Atlanta Terminal Station, Southern Railway.

inches between opposite faces. The box is about 4 feet high.

[1]"From either corner of this box extends a series of eight cantilever ribs reaching approximately to the outer edge of the slab as shown in the accompanying view. These cantilever ribs are each 14 inches wide, 4 feet deep at the box and slope off uniformly to a width of 8 inches at the top of the slab.

[1] Engineering Record, vol. 65, page 636, June 8, 1912.

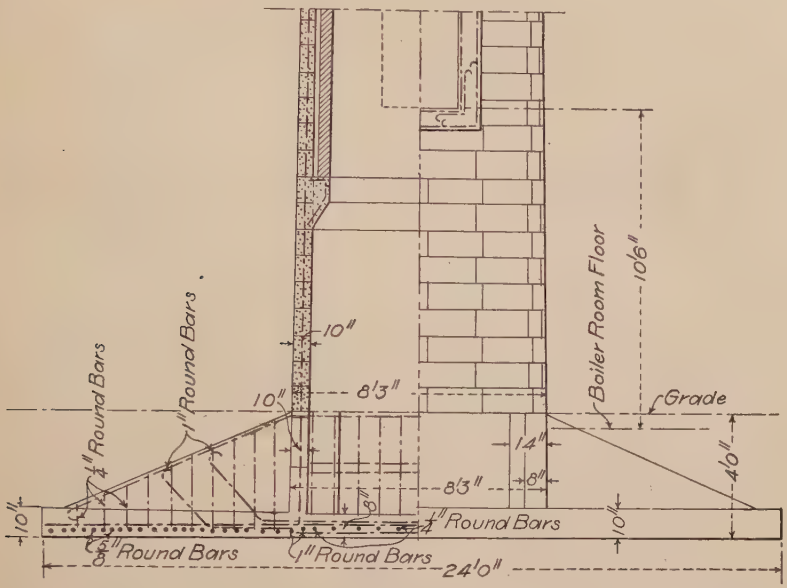


FIG. 159*b*.—Slab and Box Footing of Reinforced Concrete for a 125-foot Chimney in Chicago.

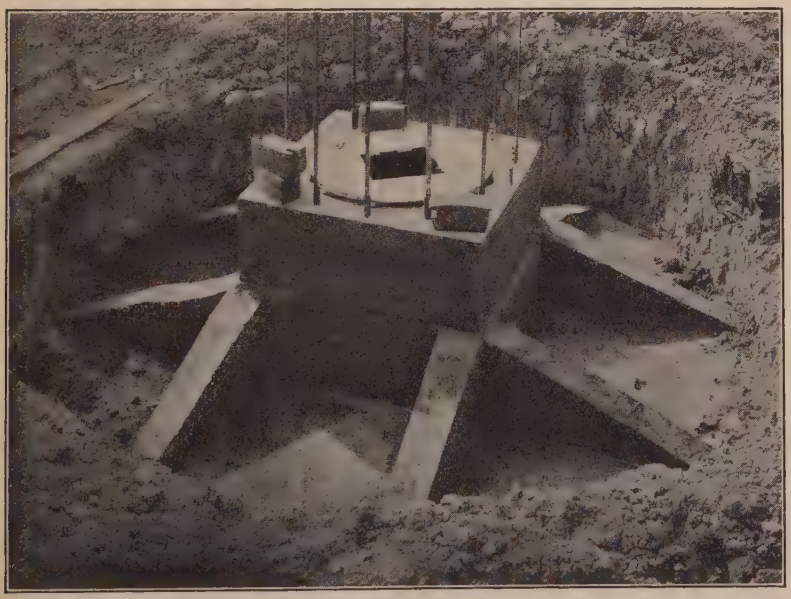


FIG. 159*c*.—View of the same Footing as shown in Fig. 159*b*.

Their effective depth is virtually 4 feet plus the effective depth of the slab, as they are built integral with it; and their reinforcement, which consists of 1-inch round bars and $\frac{1}{4}$-inch vertical stirrups, extends up from the lower surface of the slab, as shown in the accompanying drawing. The base is thus made up of a series of slabs, each supported by the adjacent cantilever ribs and reinforced with $\frac{5}{8}$-inch round bars spaced according to the position of the slab in the base. That portion of the base enclosed at the center is reinforced with a double system of $\frac{1}{4}$-inch round bars, spaced $6\frac{1}{2}$ inches on centers."

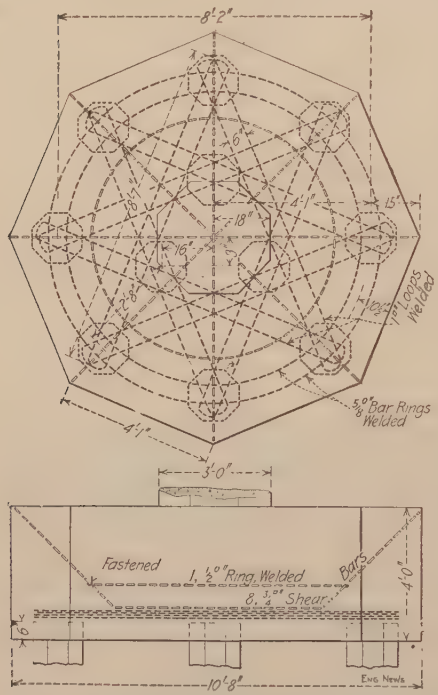


FIG. 159*d*.—Column Footing of Reinforced Concrete Supported by Pre-molded Concrete Piles.

Fig. 159*d* illustrates a reinforced-concrete footing on a pile foundation, Fig. 159*e* represents a novel type of foundation used for a loft building in New York City. There were three lines of columns, two lines of wall columns and one line through the center. The foundation presented something of a problem because the adjoining structure rested on a pile foundation, which the architect feared was in a poor condition. On account of the desire not to be forced to the expense of underpinning this adjoining building, a deep foundation was out of the question. The simple spread footing could not be used for the wall columns because of lack of space. As finally constructed, the foundation consisted of a solid framework of reinforced-concrete beams.

[1] "The special feature of the cantilever construction is that the one cross-beam and a portion of each longitudinal beam form a T-section, the center of gravity of which is the same as the center of gravity of the column loads plus the weight of the side wall. Thus, looking at Fig. 159*e*, it will be seen that half of the load coming on the column in the center of the building and the whole load coming on a wall column and the wall

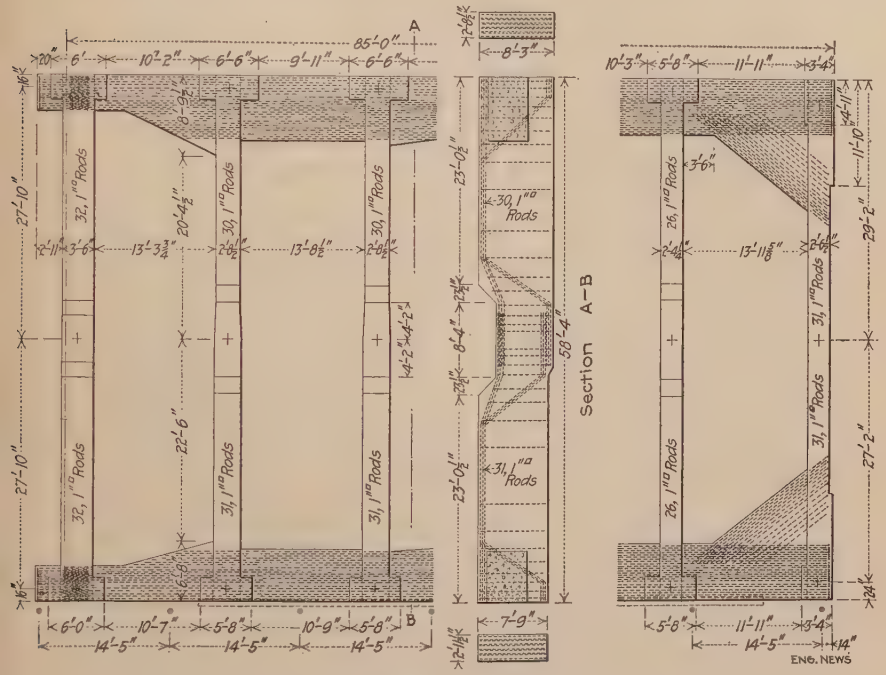


FIG. 159*c*.—Plan of Reinforced-Concrete cantilever Footings of 12-story Loft Building, 25–29 West 31st Street, New York City.

load adjacent to that column is carried on that portion of the side concrete beam and the cross-beam there shown, and that the center of gravity of these loads is the same as the center of gravity of the T-beam formed by the side beam with the transverse beam going at right angles from it. The variation in the loads and, consequently, in the centers of gravity, resulted in different shapes and sizes of the supporting beams."

[1] Engineering News, vol. 68, page 995, Nov. 28, 1912.

Reinforced-concrete spread foundations covering the whole area of the basement were used for the factories of Herman Behr & Co., and W. H. Sweeney Mfg. Co., Brooklyn, N. Y. Fig. 159*f* shows the details for the first-named factory. This raft foundation, which was of the beam-and-slab type, had a slab thickness of 1 foot and a beam thickness of 3 feet. The beams formed continuous lines under the outer wall and along the center line of the columns lengthwise of the building, the column spacing being 16 feet 10 inches longitudinally and approximately 19½ feet transversely. These beams were 5 feet wide under the

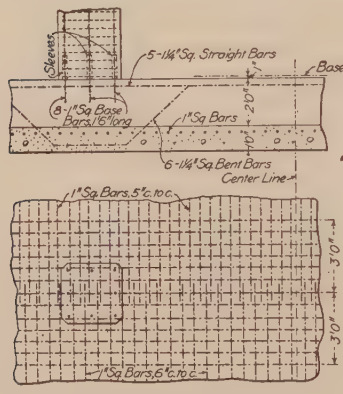


FIG. 159*f*.—Spread Foundation.

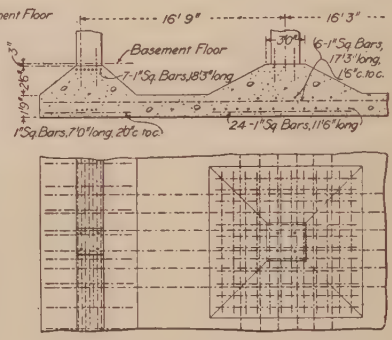


FIG. 159*g*.—Spread Foundation.

walls and 6 feet wide under the columns. The intervening space between beams was brought up nearly to surface level by a dirt fill, and a finished concrete floor was laid over the whole area. As shown in the illustration the reinforcement for the 12-inch slab consisted of transverse bars 1 inch square, spaced 5 inches on centers and 3 inches from the top of the slab. The beams under the columns were reinforced with eleven 1¼-inch square bars near the upper surface, the five center bars being carried through straight and the six outside bars bent down under the column.

The foundation of the W. H. Sweeney Mfg. Company's factory consisted of a slab over the whole area surmounted by truncated pyramidal slabs under all the columns and a trape-

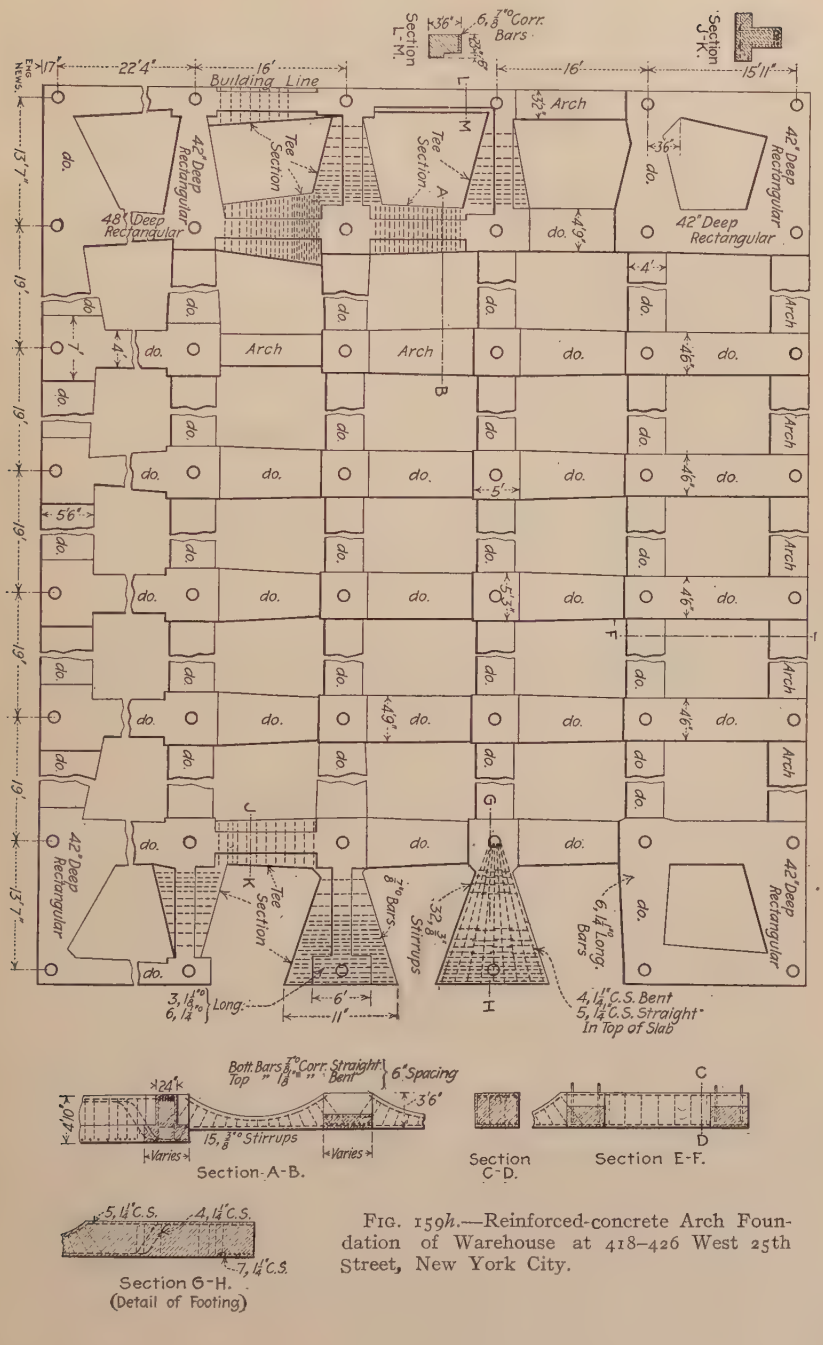


FIG. 159*h*.—Reinforced-concrete Arch Foundation of Warehouse at 418–426 West 25th Street, New York City.

zoidal-shaped slab under the wall, as shown in Fig. 159*g*. The columns were spaced approximately 16 feet on centers in both directions. The column footings were raised 2 feet 6 inches above the top of the raft slab and the latter was reinforced with six lines of rods about $1\frac{1}{2}$ feet on centers, and laid in both directions along the center lines of the columns. Further reinforcement was used in the bottom of the slab under the columns and walls, as shown in the illustration.

The inverted arch foundation of reinforced concrete as used for a building in New York City presents an unusual type of spread foundation. Its adoption was due to the necessity of having a very shallow foundation. The limit of depth fixed

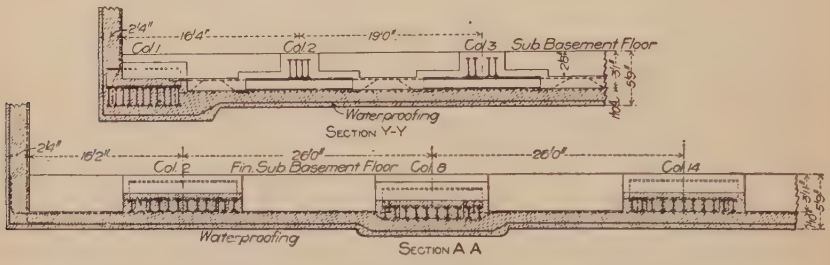


FIG. 159*i*.—Cellar Floor Sections Showing Grillage Beams and Reinforced-Concrete Girders, Pope Building, Cleveland, O.

by the architect was not sufficient for isolated reinforced-concrete footings, and as steel I-beam grillages would have cost about 25 percent more, the inverted arch form was used. The arches ran in both directions between columns as shown in Fig. 159*h*. They were 12 inches deep at the crown and 42 inches deep under the cast-iron column bases, and varied from 4 to 5 feet in width. The reinforcement consisted of $\frac{7}{8}$-inch round, straight, corrugated bars in the bottom, spaced 6 inches on centers, and $1\frac{1}{8}$-inch bent bars in the top, spaced the same distance. All end spans were made of rectangular or T-shaped concrete beams, to provide for the thrust in the adjoining arches. For further details see Engineering News, vol. 66, page 763, Dec. 28, 1911.

In the foundation for the Pope Building, Cleveland, Ohio, a

combination of a steel grillage and a reinforced-concrete raft foundation was used. The material upon which the foundation was placed consisted of a few feet of quicksand overlying clay. As the sides of the lot were enclosed by a permanent steel cofferdam extending well down into the clay, the quicksand was not subject to outside disturbance, and hence made a satisfactory cushion. A 6-inch layer of concrete was first spread over the bottom and covered with tar and felt waterproofing, after which a 16-inch layer of concrete was placed on the waterproofing. On this were located the I-beam grillages, as shown in Fig. 159*i*, section *A*–*A* being taken at right angles to the street and section *Y*–*Y* parallel with the street. The grillages were made of two tiers of 24-inch I-beams, each supporting a single column. In all the intermediate spaces the concrete floor slab was reinforced with rods, thus providing for the distribution of the column loads over the entire bottom.

CHAPTER XVI

UNDERPINNING BUILDINGS

Art. 160. Needle-Beam Underpinning

The technical term underpinning is used to denote the placing of new foundations or supports under existing structures. As an engineering art and science this work has been developed almost entirely in a few large cities, notably New York, Chicago and Boston. In New York the subways and the modern 'sky-scraper,' with its foundations carried far below those of surrounding structures, have compelled the placing of new and deeper foundations for many buildings. Some of these underpinned buildings have wall loads as high as 45 tons per linear foot and column loads of 300 tons or more. The underpinning of such heavy buildings requires great skill and care, for it must be done in such a manner that no settlement occurs; with the mechanical equipment of the modern office building, such as elevators, motors, engines, etc., a very slight differential settlement often causes trouble. Moreover, the work must often be done hastily and in a limited space.

The two general methods of underpinning are: First, the use of needle-beams to support the structure temporarily, after which the old foundations are removed and new ones placed; and second, the use of vertical cylinders (without temporarily supporting the structure) in the plane of and under the walls, carried down to solid bearing.

The needle-beam method of underpinning may be called the indirect method since the function of the needle-beams is merely to take the loads temporarily from the old foundation to permit removing the latter and the building of new foundations. This method is the older and more widely used, being universally applied where the new foundation is of a simple type and not carried to a great depth.